ADDENDUM NO. 1

TO: ALL BIDDERS OF RECORD

PROJECT: Makoshika State Park Switchback Road Repair

FWP PROJECT #: 7096421
DATE: March 12, 2014

FROM: Dax Simek, P.E., Morrison Maierle, Inc. Project Manager

Acknowledge receipt of this addendum by inserting its number and date in the Proposal Form and on the Bid Envelope. Failure to do so may subject bidder to disqualification.

This Addendum forms a part of the Contract Documents. Clarification and/or modifications area as follows:

CLARIFICATION:

1. The number of anchor blocks to be installed is 39, as shown on Plan Sheet C-5.
2. The loading for the anchor bottoms is to be established by the verification test prior to production installation.
3. The erosion control fabric shall be SC-150BN, or approved equal.

MODIFICATION:

1. The Final Geotechnical Engineering Report, dated January 31, 2014, shall be included in the Contract Documents.

Final Geotechnical Engineering Report

Makoshika State Park Road Stabilization

Dawson County, Montana

January 31, 2014

Terracon Project No. C4135319

Prepared for:

Morrison-Maierle, Inc.

Billings, Montana

Prepared by:

Terracon Consultants, Inc.

Great Falls, Montana

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Geotechnical ■ Environmental ■ Construction Materials ■ Facilities

January 31, 2014

Morrison-Maierle, Inc.
315 North 25th Street, Suite 102
Billings, Montana 59101

Attn: Mr. Dax Simek, PE, LEED AP
P: (406) 656 6000
C: (406) 451 1065
E: dsimek@m-m.net

Re: Final Geotechnical Engineering Report
Makoshika State Park Road Stabilization
Dawson County, Montana
Terracon Project Number: C4135319

Dear Mr. Simek:

Terracon Consultants, Inc. (Terracon) has completed the geotechnical engineering services for the above referenced project. This study was performed in general accordance with our original proposal number D2612197 dated August 29, 2012 which was subsequently incorporated into your contract with project owner, Montana Fish, Wildlife, and Parks Department and authorized in our contract dated July 29, 2013. This report presents the findings of the subsurface exploration and presents the results of monitoring, testing, and analysis, conducted in order to provide geotechnical recommendations for the improvement of slope and roadway performance at the referenced project. This information has been discussed and provided to you in various communications for incorporation into plan submittals as the project design has proceeded.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report, or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.

Gary A. Quinn, P.E.
Senior Geotechnical Engineer/Principal

Brian W. Evans
Senior Project Geologist

Reviewed by: Brian J. Williams, P.E., P.G.

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Dawson County, Montana
Terracon Project No. C4135319
January 31, 2014

1.0 INTRODUCTION

In 2012, a preliminary geotechnical investigation was conducted for the project by Terracon with results presented in the Geotechnical Engineering Report dated August 29, 2012. In that report, several conceptual repair options were discussed for the landslide area activated in 2011 at the roadway switchback between approximate Stations 15+50 and 17+50. The report also indicated that additional areas of instability were evident in and around this general vicinity and that further investigation was necessary to assess complexities of the landsliding and to develop final designs for stabilization of the area within risk and budgetary constraints.

In order to perform final stabilization analysis and prepare recommendations for slope repair, a supplemental subsurface investigation program was conducted for the following purposes:

- characterize subsurface conditions on a broader scale,
- sample appropriate materials for laboratory shear strength testing,
- monitor ground movements and groundwater levels.

The drilling was completed in August 2013 and consisted of three borings DH-4 through DH-6 which were advanced to depths of 36.5 to 51.5 feet. Inclinometers were installed in two of the borings, and observation wells were installed at two of the drilling locations. The boring locations are shown on Exhibit A-2, Site Plan and Geology Map, in Appendix A and monitoring results will be discussed in subsequent paragraphs along with discussions of remedial analysis and recommendations.

1.1 Project Information

ITEM	DESCRIPTION
Location	The project site is located along Radio Hill Road approximately 2.5 miles southeast of the Makoshika State Park Visitors Center near Glendive, Montana.

ITEM	DESCRIPTION
Existing Conditions	The existing road has a flexible (asphalt surface) pavement section that has undergone significant distress due to general aging effects along with cracking and differential movement associated with landsliding. The previous investigation area focused on the landsliding that occurred along the outboard side of the roadway in 2011 between approximate Stations 15+00 and 17+00.
Current ground cover	The roadway has been shifted inward through the previous slide area and remains gravel-surfaced. Vegetation on the failed slope and around the switchback area consists generally of grasses and sagebrush with some juniper trees.
Existing topography	Broken topography exists in the 2011 slide area before the terrain drops into a deeply incised coulee that has been eroded into interbedded shale and sandstone. Within the switchback area and above the roadway, numerous slide scarps exist outside the 2011 failure zone.
Proposed construction	Remedial reconstruction is proposed to reclaim and stabilize the 2011 slide area while providing a surfaced roadway on the current alignment shift. In addition, roadway surfacing and drainage improvements are planned between Stations 10+00 and 19+30 encompassing the general slide-prone area within the roadway switchback.

1.2 Landslide Description

A detailed geologic reconnaissance was conducted by our project geologist prior to the drilling investigation. The results of this reconnaissance are presented on the attached Site Plan and Geology Map, Exhibit A-2 in Appendix A, and show the 2011 slide to be part of a much larger system of multiple slides involving movements from several directions and sources within the switchback area. The sliding is occurring as an overall slope attrition process within the sandstone escarpment that rims the switchback area on the west and south; this is a long-term erosional process that has occurred, and continues to occur, as the drainage coulee along the toe-of-slope to the east evolves. Movements of the various slide masses shown on the map are taking place primarily in a northeasterly direction toward the coulee. As individual segments of the mass move outward and downward, other movements are generated in a complex, retrogressive response.

The 2011 slide appears to involve primarily silty/clayey sand colluvium (derived from the weathering of adjacent sandstone slopes) that is moving at the sand-shale interface; movement did not appear to have extended significantly into the underlying shale. This conclusion has been based on observations made during the 2012 field work which found the sandy terrain higher on this slope segment to be extremely broken and irregular as compared to uniform slope conditions noted lower in the shale portion of the slope. The trigger for this sliding is believed to be elevated groundwater due to infiltration associated with heavy spring rains in 2011. This resulted in destabilizing seepage forces exiting the slope face above the shale and uplift on the slide mass reducing resistance along the sand-shale interface. The slide conditions are shown in the following photographs:

2011 slide in sand soils in the upper reaches of the slope near Station 16+00

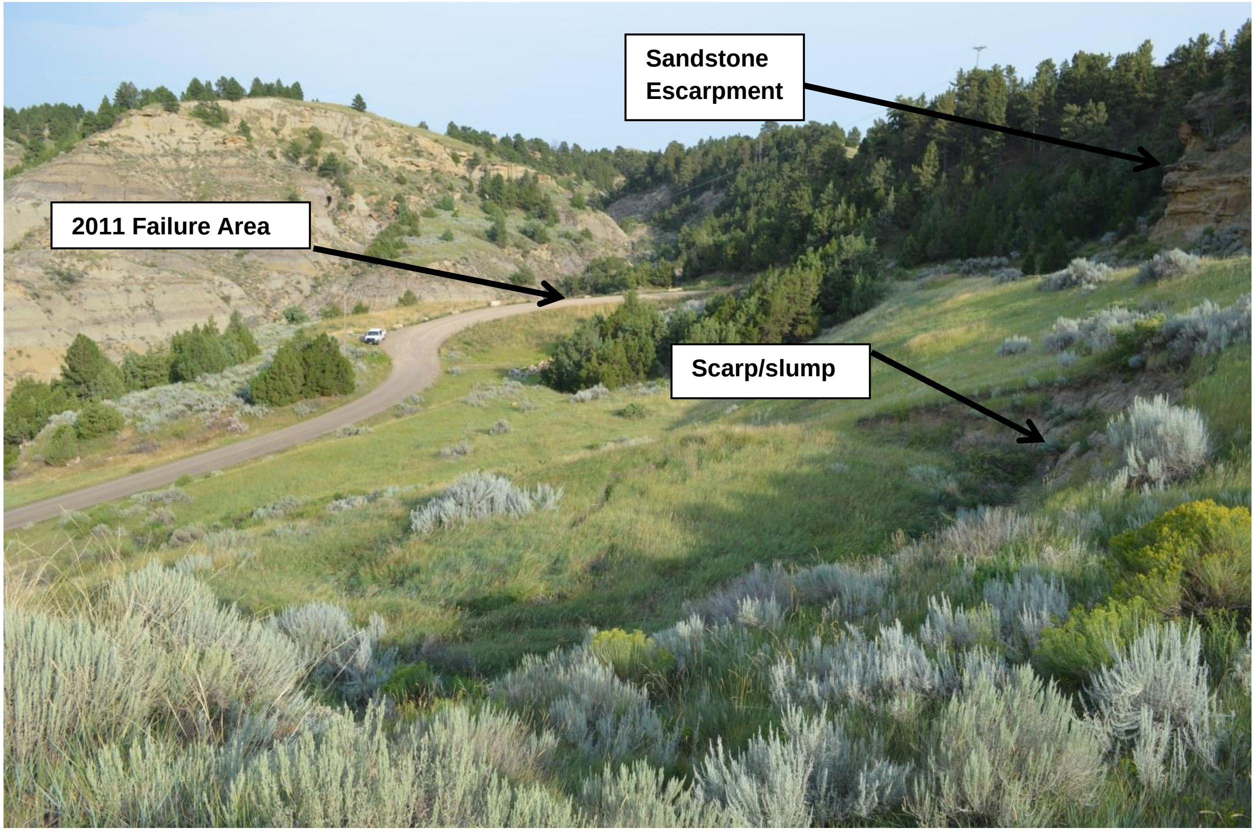
Exit of the 2011 slide at the shale interface above the coulee

The ground movements within the project limits upstation from the 2011 slide have produced slope and pavement cracking as shown in the following photographs between approximate Stations 12+00 and 15+00 where the alignment follows along the toe of a sandstone escarpment:

Small scarp above the roadway along the switchback in vicinity of Station 14+00

Cracking and differential movement in pavement in vicinity of Station 14+00

On the inside of the switchback, a prominent scarp is evident about halfway up the slope toward the crest of the hillside to the west as shown in the photograph below:



Scarp and bulge zone from recently active slump moving east inside the switchback

1.3 Subsurface Investigation

Three hollowstem auger borings (DH-4 through DH-6) were conducted at locations shown on the Site Plan (Exhibit A-2) to depths ranging from 36.5 to 51.5 feet. Borings DH-5 and DH-6 were fitted with inclinometer casing to monitor ground movement, and the casing in DH-5 was slotted to allow groundwater level monitoring. Boring DH-4 was completed with the installation of a groundwater observation well.

Standard Penetration Testing was conducted at maximum vertical intervals of 5 feet, and a California ring sampler was used at selected locations to obtain relatively undisturbed samples for laboratory shear strength testing. A more detailed description of the subsurface investigation program is provided as Exhibit A-1; Logs of Boring are also included in Appendix A.

2.0 GENERALIZED GEOLOGIC AND SUBSURFACE CONDITIONS

2.1 Surficial Geology

The surficial geology of the project area is dominated by soft rocks of the Cretaceous Hell Creek Formation and erosional sediment derived from the weathering of these rocks. The Hell Creek

Formation is described as "Dominantly gray and gray-brown sandstone, smectitic silty shale and mudstone, and a few thin beds of lignite, carbonaceous shale, and bentonitic clay/shale. Sandstones are fine or medium-grained and calcium carbonate-cemented concretions are common in the fine-grained sandstones. The beds are generally poorly cemented and weather to badland topography. Swelling clays produce characteristic popcorn weathering".(1) Information developed from the US Geological Survey Seismic Hazards website indicates the project area to be relatively aseismic with the Peak Ground Acceleration (PGA) estimated at less than 0.04g for an earthquake with a 2% probability of exceedance in 50 years, or approximately equivalent to a seismic event with a 2,475 year return interval.

(1) Open File Report No. 375, "Geologic Map of the Glendive 30' x 60' Quadrangle, Eastern Montana and Adjacent North Dakota", Montana Bureau of Mines

2.2 Soil/Rock Conditions

Conditions encountered at each boring location are described on the individual Logs of Boring found in Appendix A of this report. These descriptions combine both field logging and detailed laboratory review conducted by the geotechnical engineer and geologist. Stratification boundaries on the logs represent the approximate location of changes in soil and rock materials; in situ, the transition between materials may be gradual. The following paragraphs provide a summary discussion of the materials found in the borings.

In general, the subsurface profile consisted of 18 to 35 feet of silty to clayey sand overlying clay shale. The sand was typically loose to very loose with Standard Penetration Test (SPT) N-values often in the range of 3 to 5 blows per foot. The shale was found to be a very soft rock material with N-values of 15 to 20 in Boring DH-4, while in the remaining two borings, N-values typically exceeded 30 below the immediate bedrock contact surface. During drilling, wet conditions as noted are shown on the logs which also show the subsequent groundwater level monitoring data collected in the fall of 2013.

2.3 Inclinometer Monitoring

Initial readings for the inclinometers were recorded at the time of drilling, and comparative readings were taken subsequently on September 20, 2013 with results depicted in profile on the attached Inclinometer Summary sheets included in Appendix A as Exhibits A-6 and A-7. Deflections of the casing are plotted relative to the initial, normalized vertical "0" displacement line. The A-axis is oriented in the "Direction of Sliding" noted on the Site Plan, while the B-axis is in the perpendicular direction. The readings for DH-5 in the vicinity of the 2011 slide show movements of 0.5 to 075-inch at or near the shale surface running in the transverse or B-axis direction; this is interpreted as indicating movement mostly governed by the tendency of the mass at large to move in the northeasterly direction. The readings for DH-6 show two movement zones, one near the shale contact and another higher zone at approximately midheight in the sand overburden. It is recommended that subsequent inclinometer readings be conducted in the spring of 2014 to allow further assessment of slope behavior and any additional implications for the stabilization program.

2.4 Groundwater Conditions

Observations made during drilling noted that soils (silty or clayey sand) became wet at depths between approximately 10 and 15 feet below ground surface. Subsequent groundwater monitoring in the observation wells in September measured water levels between depths of 16 and 19 feet as noted on the Logs. These readings most likely reflect water levels depressed by the summer dry season.

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors such as localized infiltration variations. These variables have an uncertain effect on slope seepage conditions and would require long-term study with additional observation points/methods to establish thorough groundwater models for the slide areas. Therefore, groundwater levels during construction or at other times in the life of the project may be higher or lower than the levels indicated on the logs and plots. The possibility of groundwater level fluctuations has been incorporated into the design as can be reasonably approximated at this time, and construction planning should also consider the likelihood of seasonal groundwater level increases. Further groundwater monitoring is recommended in the interim between plan preparation and construction.

3.0 LABORATORY TESTING

A general description of the laboratory testing program is included as Exhibit B-1 in Appendix B. Test results are presented on Exhibits B-2 through B-8 , on the Logs, and are further discussed in the following paragraphs.

3.1 Index Properties

The sand soils had low to high plasticity fines with Liquid Limits ranging from 20 to 53%, and the clay zone at the shale interface had a Liquid Limit of 95%. This high plasticity is consistent with the 2012 investigation findings wherein the shale was found to be generally fat with Liquid Limits exceeding 80%. Plasticity in this range is generally associated with materials of low shear strength.

3.2 Shear Strength Properties

The shear strength testing program focused on determining parameters for the silty/clayey sand comprising the majority of the slide mass along with some sandy lean clay (completely weathered/residual shale) at the base of the mass at the bedrock interface. Since seepage is believed to be a primary destabilizing force, effective stress parameters were determined by drained direct shear testing to provide effective stress parameters for the stability analysis. Shear strength testing was also conducted on the clay shale. This testing included drained direct shear and undrained (quick) direct shear determinations for shale samples from Boring DH-6. The results of the testing are summarized in the following table and presented in Appendix B.

Boring/ Material	Depth (ft)	Dry Density (pcf)	Natural Moisture (%)	Peak Friction Angle Ø' (degrees)	Peak Cohesion c' (psf)	Residual Friction Angle $Ø'_r$ (degrees)	Residual Cohesion c'_r (psf)
DH-5 Clayey Sand	9.5-11.5	97	12	23	100	—	-----
DH-5 Clayey Sand	19.5–21.0	97	22	31	200	-----	-----
DH-5 Sandy Lean Clay	29.5-31.0	91	28	15	750	___	72% decrease (1)
DH-6 Clay Shale(2)	45.0-46.3	102	21	11	2250	-----	-----

(1) Based on loss in shear strength at normal pressure of 2000 psf; one residual point only
(2) Consolidated-undrained shear strength testing of a single specimen determined a shear strength of 4700 psf at a normal pressure of 2000 psf.

Unconfined compressive strength testing conducted for shale samples recovered during the 2012 investigation indicated a shear strength range of about 3,000 to 11,000 psf.

3.3 Corrosivity

Selected sand and shale samples were tested for minimum resistivity and sulfate content with the following results:

Boring/Material	Depth, ft	Resistivity, ohm-cm	Sulfate Content %	pH
DH-5 Clayey Sand	14.5-21.5	1150	0.011	9.0
DH-6 Silty Sand	25.0-26.5	350	0.45	7.4
DH-6 Clay Shale	40.0-41.5	270	0.016	9.8

These resistivity values indicate significant corrosion potential for metal in contact with the project materials. Sulfate content of the materials was also randomly high and in the range of severe attack potential for normal strength concrete. Therefore, both concrete and metal are at risk in this subsurface environment, and anchor, drain, and culvert elements should be designed for resistance accordingly.

4.0 STABILITY ANALYSIS AND REMEDIAL RECOMMENDATIONS

The analysis and remedial design have been conducted with the primary objective of stabilizing the area in and around the segment of the roadway impacted by the 2011 failure. Based on the foregoing geologic reconnaissance discussions, it has been evident from early in the investigation that the stability problems on this hillside are of significantly greater extent than merely those in the 2011 failure area. To mitigate all instabilities noted on the hillside would be economically prohibitive and require much greater investigation and remedial design efforts/costs. In consideration of these constraints, the remedial design has focused on mitigation measures that will address the 2011 failure while providing some stability improvement of the larger-scale problems that appear most likely to impact this area in the near term.

The results of our analysis, including remedial recommendations, have provided progressively via the interim report of October 29, 2013 along with numerous geotechnical design sketches and review comments for incorporation into plan submittals. In addition, specifications for key geotechnical elements such as post-tensioned anchors and horizontal drains have been prepared and provided.

4.1 Stability Analysis Methods and Rationale

Since the instabilities in this zone between approximate Stations 12+00 to 19+00 include some varying mechanisms and directions of movement, the stability analysis and remedial design have considered several methods for improving stability in the immediate switchback area which will have positive effect on the overall hillside as well. These areas of concentration are listed as follows along with summary discussions of stability problems and stabilization rationale:

- 2011 Failure Area (Stations 15+50 – 17+50)
  This area is a relatively localized portion of the unstable terrain that rises from the coulee toward the sandstone escarpment to the south. The failure appears to have occurred at the interface of the sandy colluvium and the underlying shale and have been triggered by an excess of infiltrated moisture. The sandy overburden comprising the upper portion of the slope appears to have existed at an inclination generally steeper than would have been considered sustainable based on the friction angles determined in the shear strength testing. With an elevated groundwater level above the shale and increased seepage forces, base sliding resistance along the shale interface and slope face stability in the sand would simultaneously diminish. Therefore, slope flattening and drainage are considered to be essential components for stabilization at this location.

- Stations 12+00 – 15+00
  This stretch of the roadway alignment runs along the toe of the colluvium or slope debris wasting from the sandstone escarpment. This material along the toe is marginally stable based on observations of ground and pavement cracking as well as per the inclinometer movement data. Since earthwork solutions such as buttressing might only serve to aggravate existing instabilities further downslope, mechanical anchorage of this critical segment of the roadway is considered to be an appropriate stabilization measure.

Based on these general stabilization concepts, stability analyses were conducted for both areas to validate design parameters and develop remedial designs. The stability analyses were conducted by 2-dimensional limiting equilibrium methods using the computer program GSTABL7 version 2 for both planar and circular surface review. In the limiting equilibrium approach, destabilizing forces/moments are compared to the opposing resistance and a factor of safety calculated. At a factor of safety FS = 1.0, resisting and driving conditions are equal, and failure impends. Based on normal geotechnical practice for slope stability analysis and design, it was determined that a minimum factor of safety in the range of FS = 1.3 to 1.5 should be provided for landslide remediation.

The analysis efforts were initiated by utilizing existing slope sections along with the assigned shear strength and groundwater parameters to backcalculate factors of safety for the as-is or original conditions. This was done as a reasonability check for these parameters to ascertain that the analyses of would produce factors of safety at or below FS = 1.0 consistent with the approximate location and depth of observed failure conditions.

4.2 Stability Analysis Results and Remedial Action

4.2.1 Stations 15+50 – 17+50 (2011 Failure)

The analysis was initiated with backcalculation for the existing failed slope to ascertain that a reasonable groundwater level together with the shear strength parameters determined by laboratory testing that would yield a Factor of Safety at approximately FS =1, which represents the current marginal stability condition. This check was conducted for both circular and block sliding surfaces initiating within the observed failure zone along the slope crest and exiting on the slope along or above the shale interface. This analysis was based on an idealized subsurface cross-section through Borings DH-4 and DH-5.

The shear strength along the shale surface was reduced to a magnitude between peak and residual conditions to reflect the likelihood that prior shearing had occurred in this zone and caused some strength loss. The Factors of Safety for the existing slope were determined to be on the order of FS = 1.06 and FS = 0.92 for sliding block and circular failure surfaces respectively as shown on Exhibits D-1 and D-2 in Appendix D. The groundwater level in these analyses was taken to be within ±15 feet of prevailing ground surface as was measured during the fall 2013 monitoring trip. Although this groundwater level is probably low relative to the 2011 failure condition, the analyses results were considered to reasonably validate the model, and remedial analyses were then conducted.

The remedial concept was based on maintaining the 2012 inboard roadway alignment shift as a a buffer zone separation from the 2011 failure limits to the extent possible. This would allow flattening the upper sandy portion of the slope as a prudent next step since the existing inclination of ±40º was clearly not sustainable in the loose sand. Since seepage is believed to be a major contributing factor to the instability at this location, as well as in the surrounding area of the hillside, internal drainage is recommended as a critical component of the remediation program.

The central infield area to the west (inboard) side of this roadway segment is a depression that appears to serve as a large sink for runoff which can then infiltrate the area above the slope failure. To mitigate this problem, you have indicated that this area will be regraded to positively drain to a new catch basin, and that a liner will be provided to further limit infiltration. These measures are viewed as positive measures to limit infiltration in the infield area immediately adjacent the road.

In addition, the installation of horizontal drains is recommended as a primary means to lower the groundwater level in the failure area and upslope from the infield where past slope movements are apparent. Horizontal drains are a conventional means of lowering groundwater to improve stability, and such drains are generally well-suited for sand slopes. The spatial arrangement of the drains is typically an experienced-based, empirical process and production of the drains is usually variable making the design inexact with field modifications often necessary. For purposes of analysis, it has been assumed that two tiers of drains installed in a fan pattern to cover the slope and the switchback infield will be required to reduce water levels to approximately one half of the sand thickness on the interior of the slope with progressive drawdown toward collection points at the slope face.

The tiered drains are recommended to fan from entry levels on the slope face at approximate Elevations 2345 and 2360 between projected Stations 16+25 and 16+50. The fan sweep has been set to subtend angles of 20º and 35º for the lower and upper tiers respectively. Four drain lines are recommended for the lower tier and five lines on the upper. Extension of the drains deeply upslope into the infield area to the west is believed to be beneficial for improving overall hillside stability by lowering groundwater levels. Drains of 450 to 500 feet in length at an installation angle of ±7º above horizontal are proposed for the lower tier, while much shorter drain lines on the order of 150 to 225 feet are proposed at similar inclination for the upper tier.

Stability analyses were conducted with the slope flattening and horizontal drains for both sliding block and circular failure modes. Factors of safety of FS= 1.37 and FS = 1.52 were determined for the block and circular failure surfaces respectively; these results are shown on Exhibits D-3 and D-4 which represent the concluding analyses for numerous failure scenarios. The block factor of safety is lower due to the longer failure surface generated at the shale interface where shear strength intermediate between peak and residual was input for conservatism.

4.2.2 Stations 12+00 – 15+00

Initial backculation of existing stability conditions for this slide area was conducted as described above for the 2011 slide using a subsurface cross-section through DH-4 and DH-6. Numerous circular and sliding block failure surfaces with groundwater near the slope surface were considered using the same shear strength parameters as for the 2011 analysis. The results of these analyses found factors or safety at FS = 0.98 and FS = 1.04 for circular and sliding block failure surfaces respectively, as shown on Exhibits D-5 and D-6. This outcome indicated the model to be reasonable in producing failure scarps and slump features as observed in the geologic reconnaissance.

To improve stability of this section of roadway along the toe of the sandstone escarpment, anchorage into the stable portion of the hillside below the bedrock interface was considered as a viable and positive solution for maintaining stability of the roadway section located at the top of the primary north-trending slide movement. Post-tensioned, multi-strand anchors are recommended for this application. It is further recommended that infiltration near the head of this slide be intercepted and the seepage line lowered as possible via the installation of underdrainage in conjunction with lining of the inboard ditch.

The remedial analyses were conducted based on three rows of anchors tensioned to a working capacity of 175,000 pounds each; anchor spacing within each row was set at 15 feet. Based on the anchor contribution exclusive of any underdrainage effects, the factors of safety were then found to increase to FS = 1.50 and FS = 1.62 for circular and sliding block surfaces respectively. These results are as shown on Exhibits D-7 and D-8 which again present results culminating a series of numerous stability runs.

Underdrainage along the inboard roadway ditch is recommended as a supplemental means to intercept shallow flow of moisture infiltrating from the toe area of the escarpment. The underdrain should be a minimum of 5 feet below ditch grade and discharge into the planned inlet manhole structure near Station 15+66. In addition, your proposed impermeable lining of the ditch will provide further determent to moisture infiltration in this area near the head of the slide mass.

4.3 Stabilization Elements

The stabilization concepts discussed above will require installation of anchor and drain elements to improve stability. These items are discussed further in the following paragraphs, and our geotechnical engineer has provided input for the incorporation of these items into the project design and construction documents via sketches, details, geotechnical specification preparation, and various design stage reviews.

4.3.1 Post-Tensioned, Multi-Strand Ground Anchors

Post-tensioned, multi-strand anchors extending into the clay shale (or sandstone) bedrock were selected as the primary stabilization elements for the Station 12+00 - 15+00 slide. The anchor working capacity has been set at 175,000 pounds for a bond zone extending at least 40 feet into shale. Allowable bond stress in the shale has been preliminarily estimated at 20 psi based on empirical correlations and a safety factor in the range of FS = 2.0 to 2.5; construction load testing will be required to substantiate contractor-selected anchor hole size, embedment, and methods.

Anchor blocks were designed as 9 x 9 x2 feet reinforced concrete panels sized for an allowable bearing pressure of approximately 2100 psf (FS = 2.0) for the loose sand slope material with the blocks set at an inclination of ±60º above horizontal. For this allowable bearing value, it is recommended that a freeboard of 2 feet be maintained at the top of the block cut.

Corrosion protection is a critical requirement for the longevity of the anchors. The ground anchors and appurtenances should have Class I Corrosion Protection in accordance with the current PTI *Guide Specification for Post-Tensioning Materials* (1). Type V cement should be used in all block concrete and anchor grout.

1. Recommendations for Prestressed Rock and Soil Anchors, Post-Tensioning Institute, 2004

4.3.2 Horizontal Drains

Horizontal drains consisting of 1.5-inch diameter, Schedule 80 factory slotted PVC pipe have been selected as a stabilizing element in addition to slope flattening for the slide between Stations 15+50 and 17+50. A slot size of 0.010 inch has been selected based on standard filter criteria and the grain size of the predominant sand materials comprising the slope.

4.4 Earthwork

The primary earthwork elements for this project will include grading associated with flattening of the slope for the Station 15+50 – 17+50 slide, finished grading to cover the anchor blocks for the Station 12+00 – 15+00 slide, subgrade preparation, and backfilling for underdrain and other trenching. It is anticipated that the earthwork and backfilling will be done primarily with on-site sand materials in accordance with the following table and per approval by the Project Manager or Geotechnical Engineer. Engineered fill to be used for project earthwork should meet the material property and compaction requirements recommended in the following sections.

4.4.1 Material Requirements

On-site sand and clay soils[1,2]	SC, SM,CL (and dual symbols)	Slope, subgrade,site fill and trench backfill.
Open-graded drainage aggregate[3]	GP	Underdrain trench backfill

1. Soils should consist of approved materials that are free of organic matter and debris, and do not include soft, degradable, or deleterious particles. Frozen material should not be used, and fill should not be placed on a frozen subgrade. Each proposed fill material type should be sampled and evaluated by the geotechnical engineer prior to its delivery and/or use.
2. Moisture conditioning of soils should be anticipated for proper compaction; this may require mechanical reduction in clump size (disking, etc.) to a maximum 1-inch dimension to facilitate moisture conditioning; the necessary moisture adjustment will be difficult during wet/cold seasons.
3. Open-graded drainage aggregate material should be comprised of hard, durable gravel particles with 100% passing a 3/4-inch screen and not more than 5% passing a 3/8-inch screen.

4.4.2 Compaction Requirements

Item	Description
Fill Lift Thickness	9 inches or less in loose thickness when heavy, self-propelled compaction equipment is used. 6 inches in loose thickness when hand-guided equipment (i.e. jumping jack or plate compactor) is used.
Minimum Compaction Requirement [1] **(ASTM D698)**	Slope face, slope repair, trench backfill, anchor block backfill: 95%
Moisture Content [2] **(ASTM D698)**	±2% of optimum

1. We recommend that each lift of fill be observed and tested by Terracon for moisture content and compaction prior to the placement of additional material. Should the results of the in-place density tests indicate the specified moisture or compaction limits have not been met, the area represented by the test should be reworked and retested until the specified moisture and compaction requirements are achieved.
2. Moisture conditioning of the native sand and clayey soils will be required for proper compaction.

4.4.3 Slope Grading Requirements

- The slope flattening for the Station 15+50 – 17+50 slide must also involve the repair of all cracked or otherwise unstable ground associated with the previous sliding. This work should be conducted under the observation and per the approval of the Geotechnical Engineer.

- The finished slope cover of the anchor blocks for the Station 12+00 – 15+00 slide should be no steeper than 2H:1V.

- Topsoil should be salvaged from all excavation work for finish slope dressing. Seed mixtures should be developed in concert with the Owner to provide a vegetative cover compatible with the project environment.

5.0 FINAL DESIGN AND CONSTRUCTION CONSIDERATIONS

5.1 Final Design Review

- It is recommended that our Geotechnical Engineer conduct a final plan-in-hand review of the site with you to ascertain constructability and compatibility of recommended stabilization measures with project constraints. Final plan and design review are essential components of continued geotechnical involvement prior to construction. In addition to the numerous geotechnical challenges associated with the slide repair design, there are significant construction challenges that will require owner, engineer, and contractor ingenuity and cooperation as variables in ground and environmental conditions become more apparent. It is recommended that our Geotechnical Engineer remain directly involved with the construction to assess ground conditions and construction operations relative to design assumptions. This is a critical element in transitioning from design to completion for a project of this nature in which there will undoubtedly be ground and climate variables that impact the design and construction.

- It is anticipated that the geotechnical instrumentation installed for this investigation will be lost during the remedial construction. Prior to the initiation of the construction, it is therefore recommended that one or two further monitoring cycles be conducted to allow further assessment of design assumptions and to determine the potential need for construction adjustment or advisory.

5.2 Construction Considerations

- Construction sequencing is an initial and very significant consideration in affecting the repair work. First, coordination between the general roadway contractor and the slide repair contractor(s) must be carefully orchestrated for efficiency in implementing both major components of the stabilization. The contractor should submit a work plan/schedule prior to beginning construction for review and comment by the Project Manager and Geotechnical Engineer.

- Temporary stability and construction access to the slide areas is an important consideration. The contractor is responsible for the planning, safe construction, and proper reclamation of temporary access trails, benches, and pads in accordance with OSHA regulations, prudent construction practice, and project plans/specifications. Provisions for access area stabilization may require importing of select granular aggregates and the use of geotextile/geogrid materials to maintain access for specific construction equipment and procedures. Proper surface drainage of the work areas is also necessary to improve and maintain required access and stability.

- Seepage conditions will impact the grading efforts to some uncertain extent depending on seasonal and longer-term moisture conditions. Evaluation and mitigation of seepage effects will require specific geotechnical evaluation at the time of construction. Some grading adjustments should be anticipated where wet conditions due to precipitation or seepage occur.

- Repair of the Station 15+50 – 17+50 slide should be initiated with the required regrading of the slope to approximate plan configuration. The grading efforts should also focus on closing cracks, eliminating scarps and sags, and compacting the ground surface to minimize the potential negative effects of moisture infiltration. Successful grading and compacting will most likely require blading and kneading-type compactors as a minimum to breakdown the site soils and adjust moisture to the proper compacting range as recommended herein.

- The drilling necessary for the installation of the horizontal drains and the post-tensioned anchors will encounter some difficult conditions as sandstone boulders/slabs are likely within the colluvial (slope debris) deposits. Additional difficulties with drill hole instability should also be anticipated due to loose sand and groundwater. The contractor should be prepared for drill hole advancement by such means as, but not limited to, casing, down-hole hammering and coring should be anticipated.

- Provisions for post-construction geotechnical monitoring should be implemented in each of the slide areas as a continuation of the design investigation monitoring discussed in Section 5.1 above. It is recommended that at least one inclinometer and two observation wells be installed in each slide area for post-construction monitoring. Monitoring plans should be developed for geotechnical evaluation of the repair performance after construction for long-term evaluation of slope behavior. This is probably most important with regard to assessing the impact that horizontal drains have on the groundwater conditions. If sufficient drawdown is not occurring, then the stability will not be as calculated, and the potential need for additional drain installation(s) would exist. For both slides, some post-construction slope movements should be anticipated as the stabilization elements transfer load and groundwater levels adjust.

- Load testing of anchors is recommended as an integral part of the construction processes for the assessment of anchor capacity and design compliance; our Geotechnical Engineer should remain closely involved with this testing to ascertain design assumptions and safety factors are reasonably consistent with actual conditions. Anchor block design has been predicated on a nominal 2000 psf bearing pressure for design purposes; grading and weather conditions will impact this assumption and the available ground reaction for anchor tensioning.

5.3 Maintenance Considerations

- The horizontal drains will require periodic maintenance as reduced flow capacity or possibly clogging can occur due to a variety of conditions. The drain flows should be measured routinely in conjunction with monitoring the recommended observations wells and keeping precipitation records. This information should be reviewed by our Geotechnical Engineer over the initial several years of service to assess patterns and the potential need for cleaning of the wells, a process usually accomplished by jetting.

- Post-construction monitoring of the recommended inclinometers and observation wells should be conducted by our geotechnical staff with results reviewed by our Geotechnical Engineer to assess repair performance and the need for any maintenance or additional stabilization measures.

6.0 GENERAL COMMENTS

The analysis and recommendations presented in this report are based upon the data obtained from the borings performed at the indicated locations and from other information discussed in this report. This report does not reflect variations that may occur between borings, across the site, or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

The scope of services for this project does not include, either specifically or by implication, any environmental or biological assessment of the site or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

This report has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either expressed or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Terracon reviews the changes and either verifies or modifies the conclusions of this report in writing.

APPENDIX A
FIELD EXPLORATION

Field Exploration Description

The borings were laid out by Terracon based on geologic reconnaissance conducted for this investigation. Ground surface locations and elevations of the borings were interpolated between contours of the Morrison-Maierle topographic survey.

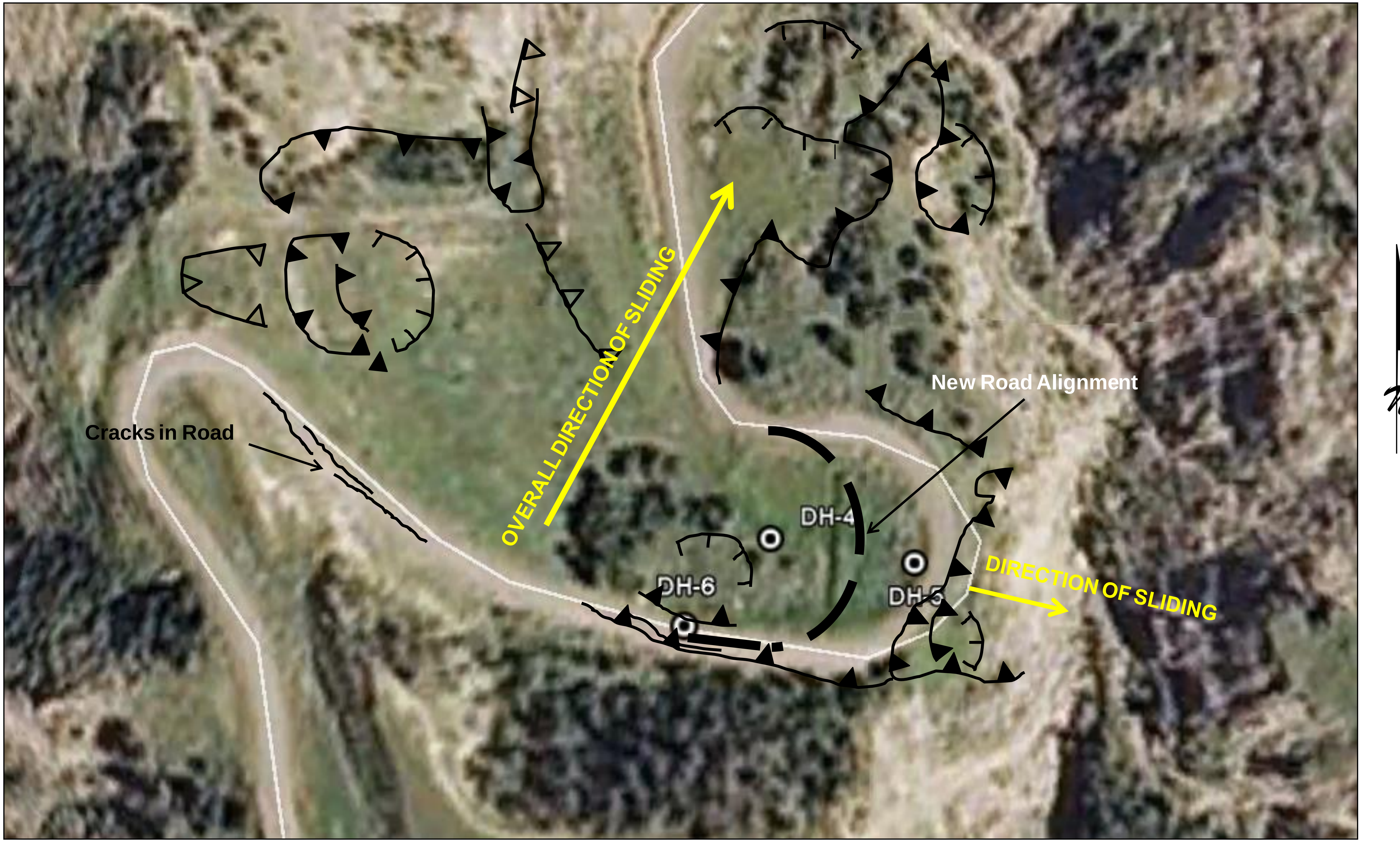
The borings were drilled with a track-mounted CME 75 truck-mounted drill rig using hollow stem augers. Samples of the soils/soft rocks encountered in the borings were obtained by driving split spoon and California samplers, and collecting auger cuttings.

In the split-barrel sampling procedure, the number of blows required to advance a standard 2-inch O.D. split-barrel sampler the last 12 inches of the typical total 18-inch penetration by means of a 140-pound hammer with a free fall of 30 inches, is the standard penetration resistance value (SPT-N). This value is used to estimate the in-situ relative density of cohesionless soils and consistency of cohesive soils.

A CME automatic SPT hammer was used to advance the split-barrel sampler in the borings performed on this site. A significantly greater efficiency is achieved with the automatic hammer compared to the conventional safety hammer operated with a cathead and rope. This higher efficiency has an appreciable effect on the SPT-N value. The effect of the automatic hammer's efficiency has been considered in the interpretation and analysis of the subsurface information for this report.

The samples were tagged for identification, sealed to reduce moisture loss, and taken to our laboratory for further examination, testing, and classification. Information provided on the logs attached to this report includes soil and rock descriptions, consistency evaluations, boring depths, sampling intervals, and groundwater conditions. The borings were completed with inclinometers or open piezometers.

Field logs were prepared by the field geologist/engineer. The logs included visual classifications of the materials encountered during drilling as well as the engineer's interpretation of the subsurface conditions between samples. The final logs included with this report represent the engineer's interpretation of the field logs and includes modifications based on laboratory observations and tests of the samples.



Approximate Boring Location
Active Scarp
Inactive/Dormant Appearing Scarp

Project Manager: GAQ	Project No. C4135319
Drawn by: BE	Scale: NA
Checked by: GAQ	File Name:
Approved by:	Date: 10/28/13

Terracon
1392 13th Ave. SW, Great Falls, MT
PH. (406) 453-5400 FAX. (406) 761-6655

Site Plan and Geology Map
Mikoshika State Park
Dawson County, MT

Exhibit
A-2

BORING LOG NO. DH-4

PROJECT: Makoshika State Park Road

CLIENT: Morrison Maierle Inc.
Billings MT

SITE: Makoshika State Park
Glendive, MT

LOCATION See Exhibit A-2

Surface Elev.: 2386 (Ft.)

DEPTH	MATERIAL DESCRIPTION	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	TEST TYPE	COMPRESSIVE STRENGTH (psf)	STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
0.2	**TOPSOIL**	2386				1-2-3 N=5				7			
3.5	**SILTY CLAYEY SAND (SC-SM)**, brown, loose, friable, roots	2382.5				3-2-3 N=5				9			
	SILTY SAND (SM), brown, loose, bedded, trace clay		5			3-2-2 N=4				17			
10.5		2375.5	10	▽		2-2-3 N=5				26			
	CLAYEY SAND (SC), brown, loose to medium dense, bedded; scattered coal and shale fragments		15	▽		3-5-5 N=10				27			
18.0		2368											
	CLAY SHALE, tan to brown, very soft rock, completely to severely weathered, poorly bonded, blocky, residual bedding and fissility, high plasticity		20			5-8-12 N=20				26			
	becomes dark grey @ 26.0'		25			4-5-10 N=15				24			
	coal fragments and lenses of light grey shale, very blocky @ 30.0'		30			6-8-11 N=19				20			
36.5		2349.5	35			4-8-11 N=19				24			

Boring Terminated at 36.5 Feet

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic
Logged By B. Evans

Advancement Method:
4 1/4" Hollowstem Augers

Abandonment Method:
installed 1.0" slotted PVC piezometer

See Exhibit A-3 for description of field procedures.

See Appendix B for description of laboratory procedures and additional data (if any).

See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

▽ *Wet; 8/22/13*

▽ *9/20/13*

Terracon

Boring Started: 8/22/2013	Boring Completed: 8/22/2013
Drill Rig: CME 55	Driller: Ryan G.
Project No.: C4135319	Exhibit: A-3

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL C4135319 MIKOSHIKA.GPJ TERRACON2012.GDT 1/31/14

BORING LOG NO. DH-5

PROJECT: Makoshika State Park Road

CLIENT: Morrison Maierle Inc.
Billings MT

SITE: Makoshika State Park
Glendive, MT

LOCATION See Exhibit A-2

Surface Elev.: 2385 (Ft.)

DEPTH	MATERIAL DESCRIPTION	ELEVATION (Ft.)
0.1	**TOPSOIL**	2385
1.0	**FILL, CLAYEY SAND (SC)**, brown, medium dense, friable, scattered gravel	2384
5.0	**SILTY, CLAYEY SAND (SC-SM)**, brown, loose, friable, roots, salts, bedded	2380
8.5	**SANDY LEAN CLAY (CL)**, dark brown, medium stiff, fine sand, bedded	2376.5
	CLAYEY SAND (SC), brown, loose to medium dense, interbedded with lenses of silty sand and poorly graded sand, fine sand	
	possible shear zone @ 15.0' +/-	
28.0	completely weathered sandstone appearance, very soft @ 25.2	2357
30.2	**SANDY FAT CLAY (CH)**, mottled brown/grey, stiff, layered, numerous shale fragments, some sandstone gravel,	2355
	CLAY SHALE, dark grey to light grey, very soft rock, severely weathered, poorly bonded, bedded, fissile, high plasticity	
	high grey bentonitic clay @ 32.0-35.0'	
	becoming very sandy, lower plasticity, blue grey @ 38.0'	
	high plasticity clay shale @ 44.5'	
51.0	blue/grey color @ 50.0', high plasticity, some very fine sand	2334
	Boring Terminated at 51.5 Feet	

DEPTH (Ft.)	FIELD TEST RESULTS	TEST TYPE	COMPRESSIVE STRENGTH (psf)	STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
	2-7-5 N=12				5			
	4-3-2 N=5				11			
5	4-3-4 N=7				15			
10	7-6-6						25-20-5	32
	3-2-3 N=5				13			
15	1-2-1 N=3				22			
20	3-4-6						53-14-39	34
	2-2-2 N=4				27			
25	3-5-6 N=11				27			
30	4-6-9				28	91	95-18-77	86
	5-7-10 N=17				38			
35	10-13-15 N=28				25			
40	16-24-32 N=56				20			
45	13-24-36 N=60				23			
50	16-24-30 N=54				18			

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic
Logged By B. Evans

Advancement Method:
4 1/4" Hollowstem Augers

Abandonment Method:
Inclinometer installed

See Exhibit A-3 for description of field procedures.

See Appendix B for description of laboratory procedures and additional data (if any).

See Appendix C for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

Wet; 8/21/13

9/20/13

Terracon

Boring Started: 8/21/2013	Boring Completed: 8/21/2013
Drill Rig: CME 55	Driller: Ryan G.
Project No.: C4135319	Exhibit: A-4

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL C4135319 MIKOSHIKA.GPJ TERRACON2012.GDT 1/31/14

BORING LOG NO. DH-6

PROJECT: Makoshika State Park Road

CLIENT: Morrison Maierle Inc. Billings MT

SITE: Makoshika State Park Glendive, MT

LOCATION See Exhibit A-2

Surface Elev.: 2418 (Ft.)

Depth	Description	Elevation (Ft.)
0.2	**ASPHALT**, 2.5' total thickness	2418
3.3	**FILL, CLAYEY SAND (SC)**, brown, loose to medium dense, scattered sandstone gravel	2414.5
35.5	**SILTY SAND (SM)**, brown, loose to medium dense, bedded, trace clay, inter-layered with beds of clayey sand; scattered coal and shale fragments @ 12.0-13.0'; very sandy, very fine coal fragments, scattered clay clods @ 25.3'; very clayey @ 31.0'	2382.5
38.0	**SANDSTONE**, grey, very soft rock, severely to completely weathered, poorly to uncemented, bedded, coal fragments, scattered shale lenses	2380
51.5	**CLAY SHALE**, grey to dark grey, very soft rock, severely weathered, poorly bonded, fissured, bedded, fissile, high plasticity, some sand, dark grey, detrital material on laminae @ 50.0'	2366.5

Boring Terminated at 51 Feet

Depth (Ft.)	Water Level Observations	Field Test Results	Test Type	Compressive Strength (psf)	Strain (%)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits LL-PL-PI	Percent Fines
2.5–4		7-6-3 N=9				13			
5–6.5		1-2-1 N=3				14			
10–11		5-4-5							
11–13		2-2-2 N=4				17			
14	▽								
15–16.5		2-2-1 N=3				22			
20–21		2-2-4							
21–23		2-1-2 N=3				16			
25–26.5		1-2-1 N=3				16			
30–31		2-5-7				25			
31–33		2-4-7 N=11				25			
35–36.5		9-14-17 N=31				22			
40–41.5		9-15-21 N=36				24			
45–46		95/0.8'				19 / 21	101		
50–51.5		11-21-29 N=50				19			

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic
Logged By B. Evans

Advancement Method:
4 1/4" Hollowstem Augers

Abandonment Method:
Inclinometer installed

See Exhibit A-3 for description of field procedures.

See Appendix B for description of laboratory procedures and additional data (if any).

See Appendix C for explanation of symbols and abbreviations.

Notes:

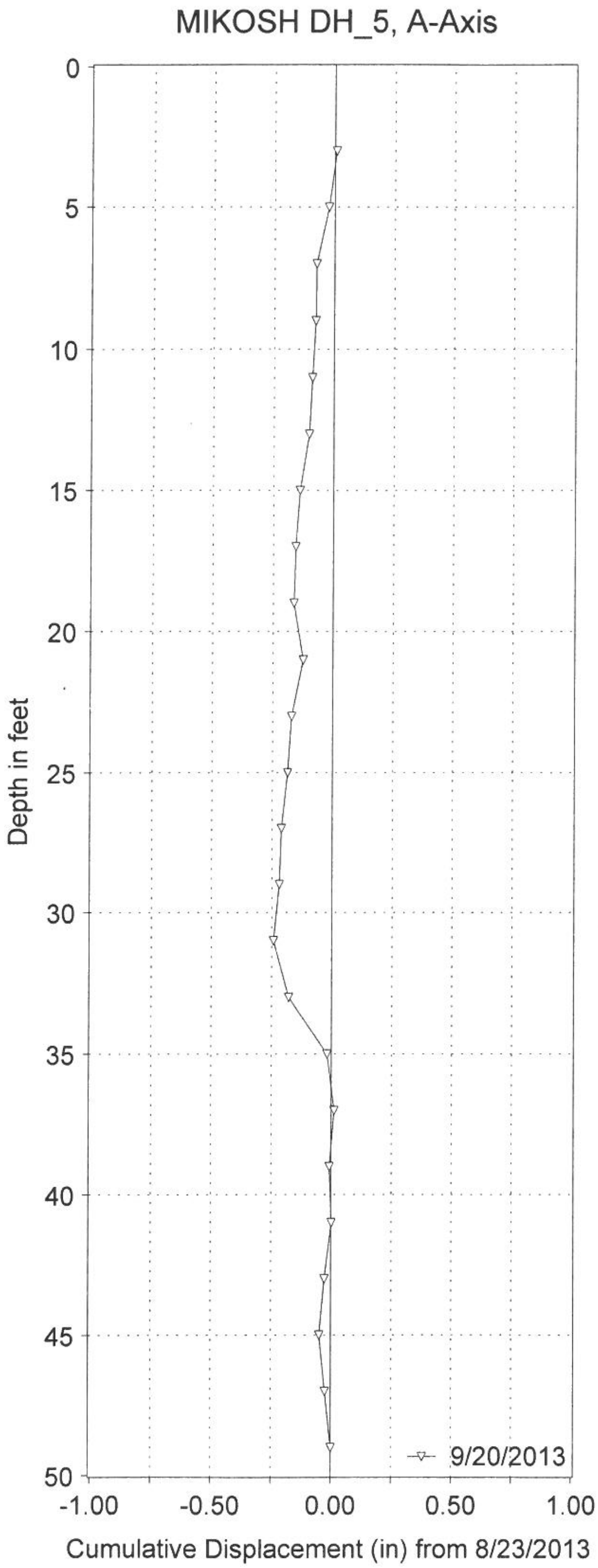
WATER LEVEL OBSERVATIONS

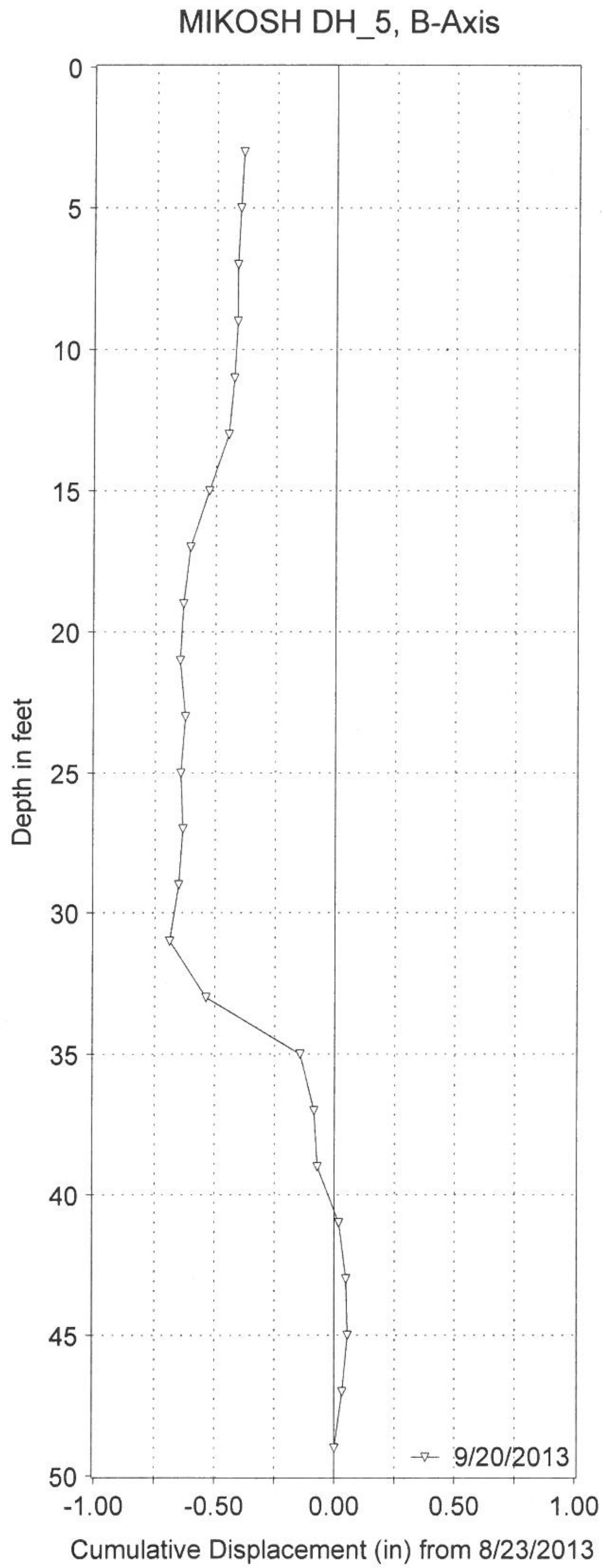
▽ *Wet; 8/22/13*

Terracon

Boring Started: 8/22/2013	Boring Completed: 8/22/2013
Drill Rig: CME 55	Driller: Ryan G.
Project No.: C4135319	Exhibit: A-5

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL C4135319 MIKOSHIKA.GPJ TERRACON2012.GDT 1/31/14





*0.0' elevation is top of casing

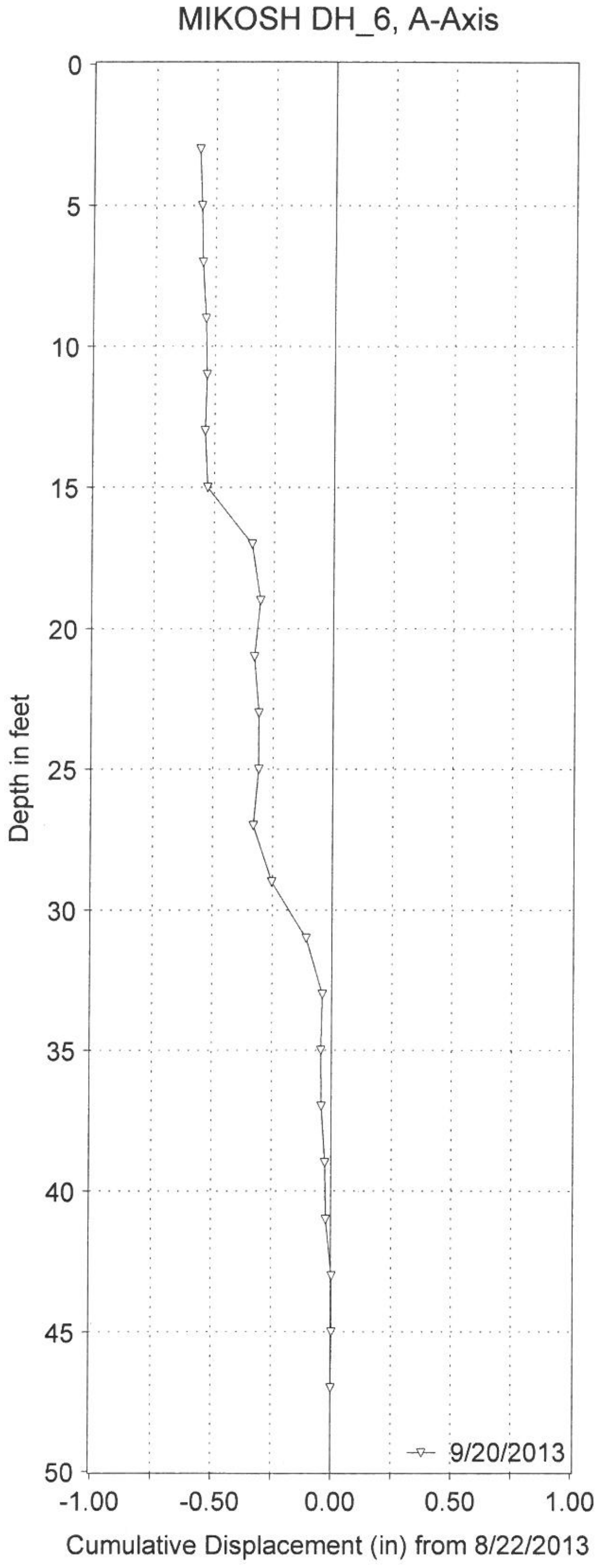
Project: Makoshika State Park

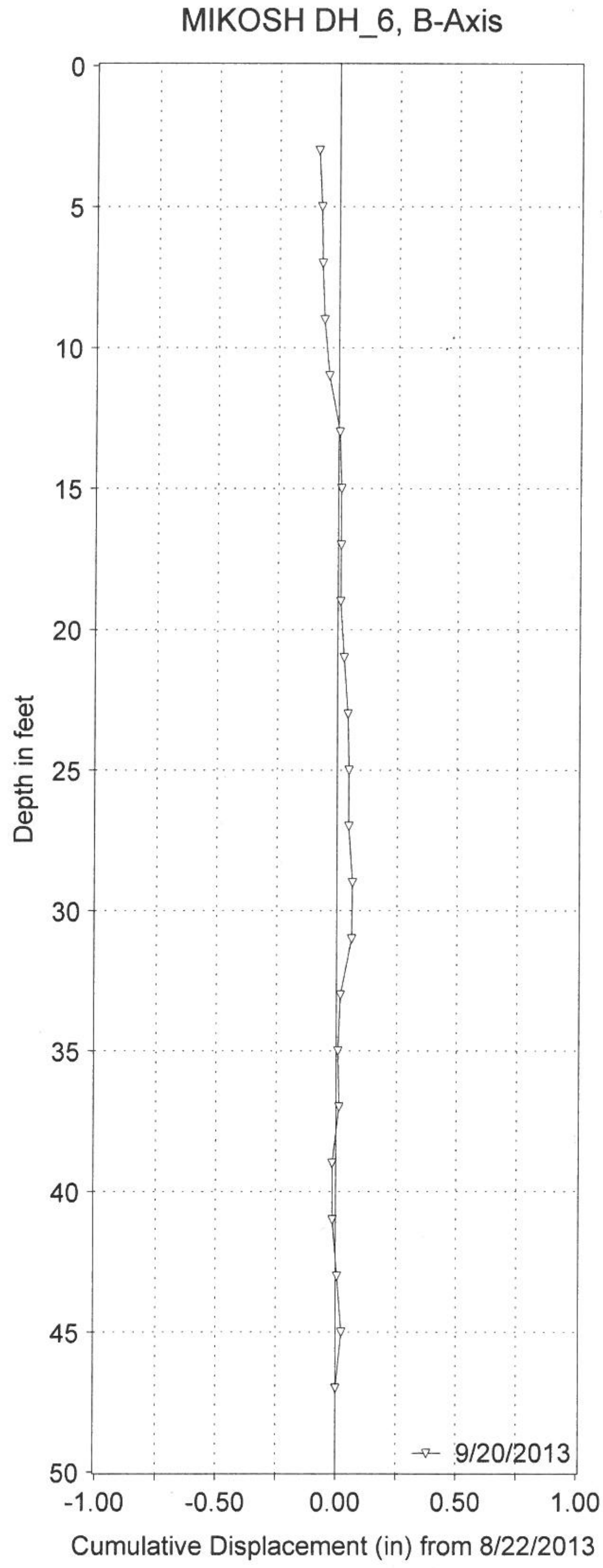
Job Number: C4135319

Terracon

Inclinometer Summary
NTL Engineering & Geoscience
Great Falls, MT

Exhibit A-6





*0.0' elevation is top of casing

Project: Makoshika State Park

Job Number: C4135319

Terracon

Inclinometer Summary
NTL Engineering & Geoscience
Great Falls, MT

Exhibit A-7

APPENDIX B
LABORATORY TESTING INFORMATION

Laboratory Testing

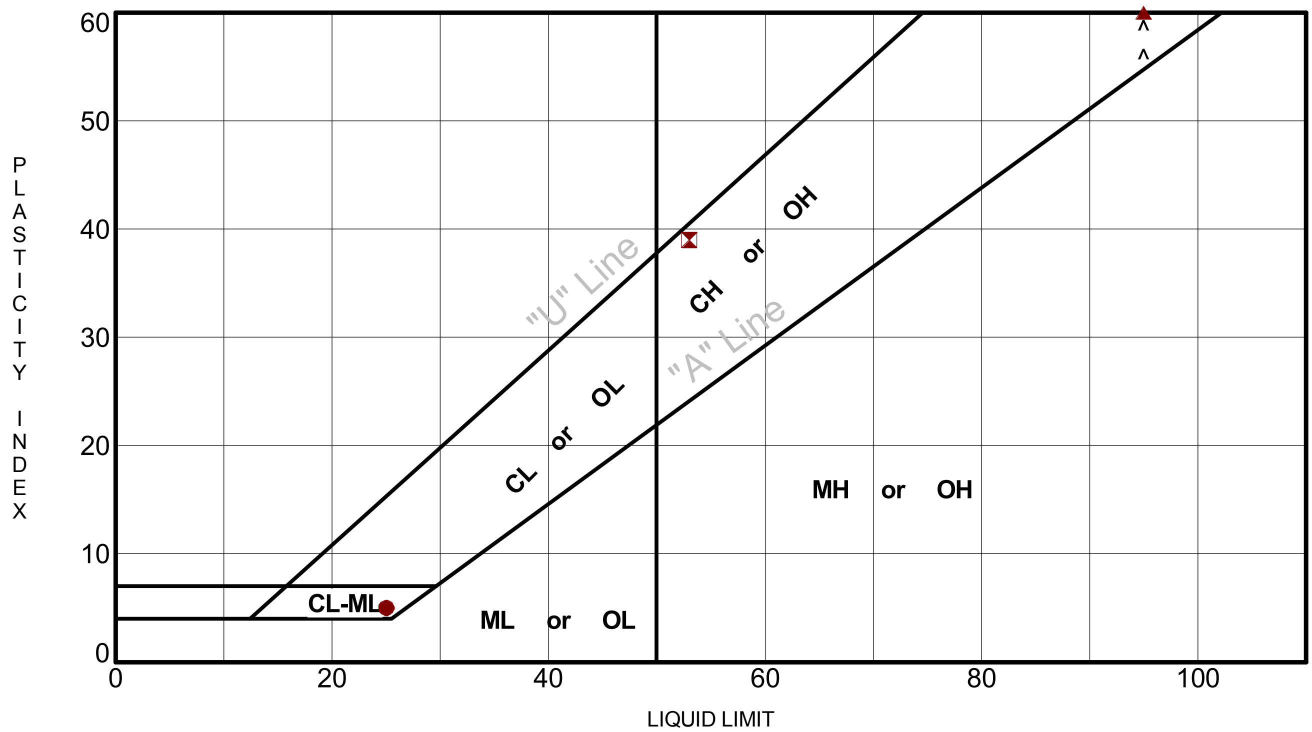
As a part of the laboratory testing program, the soil/rock samples were classified in the laboratory based on visual observation, texture, plasticity, and the laboratory testing performed as noted below. The soil descriptions presented on the boring logs are in accordance with our enclosed General Notes and Unified Soil Classification System (USCS. The estimated group symbol for the USCS is also shown on the logs, and a brief description of the Unified System is included in this report. Rock samples were visually reviewed by our geologist and geotechnical engineer to verify field descriptions, provide classification consistent with locally accepted practice for rock-like materials, and select appropriate samples for testing. Results of the laboratory tests are presented on the logs and/or included herein.

Selected soil and clay shale samples were tested for the following properties:

- Water Content
- Grain Size Distribution
- Liquid and Plastic Limits
- pH, Resistivity, Sulfate
- Drained Direct Shear
- Undrained (Quick) Direct Shear

ATTERBERG LIMITS RESULTS

ASTM D4318



Boring ID	Depth	LL	PL	PI	Fines	USCS	Description
DH-5	9.5	25	20	5	32	SC-SM	SILTY, CLAYEY SAND
DH-5	19.5	53	14	39	34	SC	CLAYEY SAND
DH-5	29.5	95	18	77	86	CH	FAT CLAY

PROJECT: Makoshika State Park Road

SITE: Makoshika State Park
Glendive, MT

Terracon

PROJECT NUMBER: C4135319

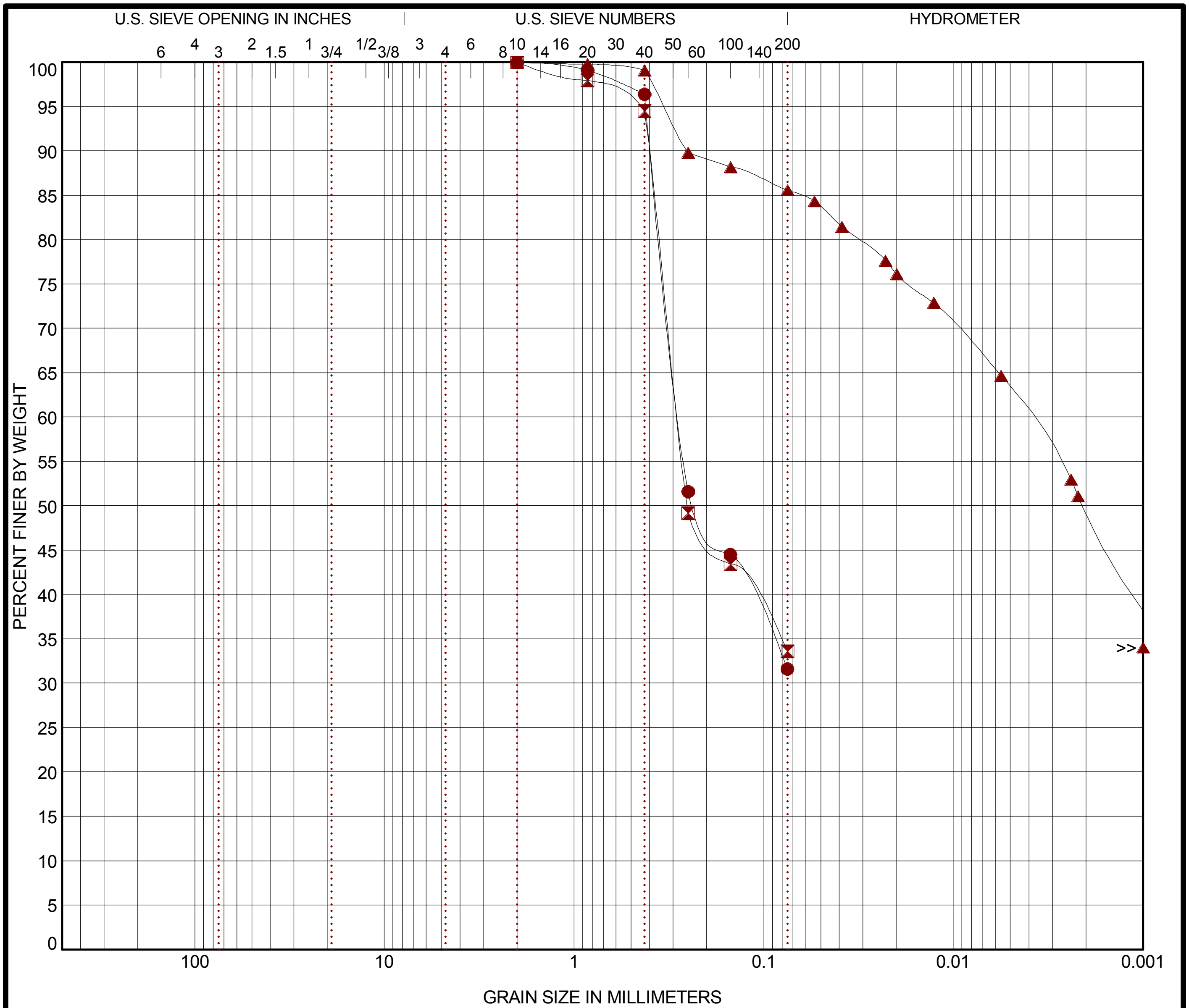
CLIENT: Morrison Maierle Inc.
Billings MT

EXHIBIT: B-2

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. ATTERBERG LIMITS C4135319 MIKOSHIKA.GPJ TERRACON2012.GDT 1/31/14

GRAIN SIZE DISTRIBUTION

ASTM D422



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring ID	Depth	USCS Classification	LL	PL	PI	Cc	Cu
●	DH-5	9.5	SILTY, CLAYEY SAND(SC-SM)	25	20	5		
⧗	DH-5	19.5	CLAYEY SAND(SC)	53	14	39		
▲	DH-5	29.5	FAT CLAY(CH)	95	18	77		

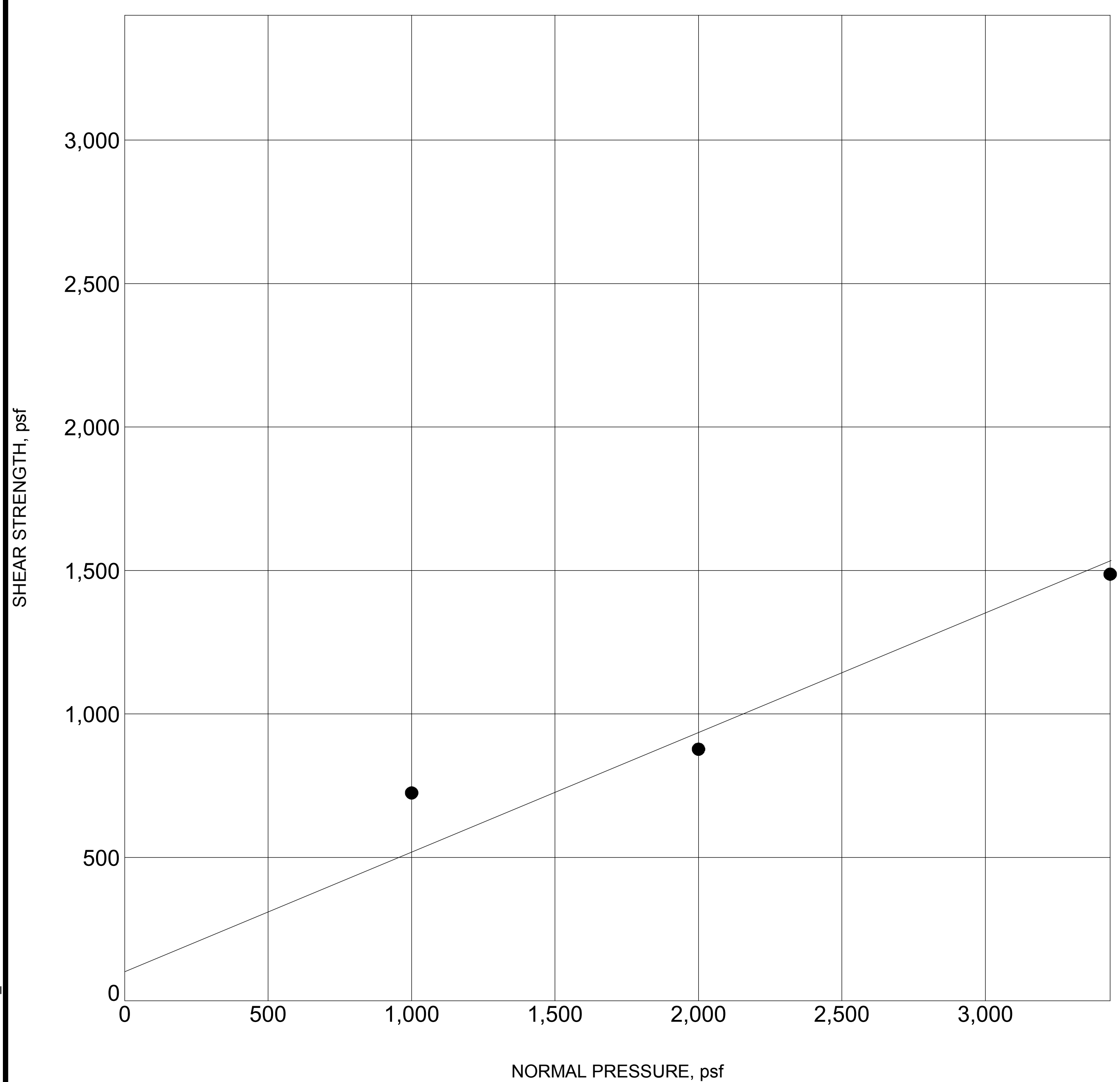
	Boring ID	Depth	D_{100}	D_{60}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
●	DH-5	9.5	2	0.276			0.0	68.4	31.6	
⧗	DH-5	19.5	2	0.284			0.0	66.4	33.6	
▲	DH-5	29.5	2	0.004			0.0	14.4	22.5	63.1

PROJECT: Makoshika State Park Road	Terracon	PROJECT NUMBER: C4135319
SITE: Makoshika State Park Glendive, MT		CLIENT: Morrison Maierle Inc. Billings MT
	,	EXHIBIT: B-3

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS-2 C4135319 MIKOSHIKA.GPJ TERRACON2012.GDT 1/31/14

DIRECT SHEAR TEST

ASTM D3080



	Specimen Identification		Classification	γ_d, pcf	WC,%	c, psf	ϕ°
●	**DH-5**	**9.5ft**	**SILTY, CLAYEY SAND(SC-SM)**	**97**	**12**	**100**	**23**

PROJECT: Makoshika State Park Road

SITE: Makoshika State Park
Glendive, MT

Terracon

PROJECT NUMBER: C4135319

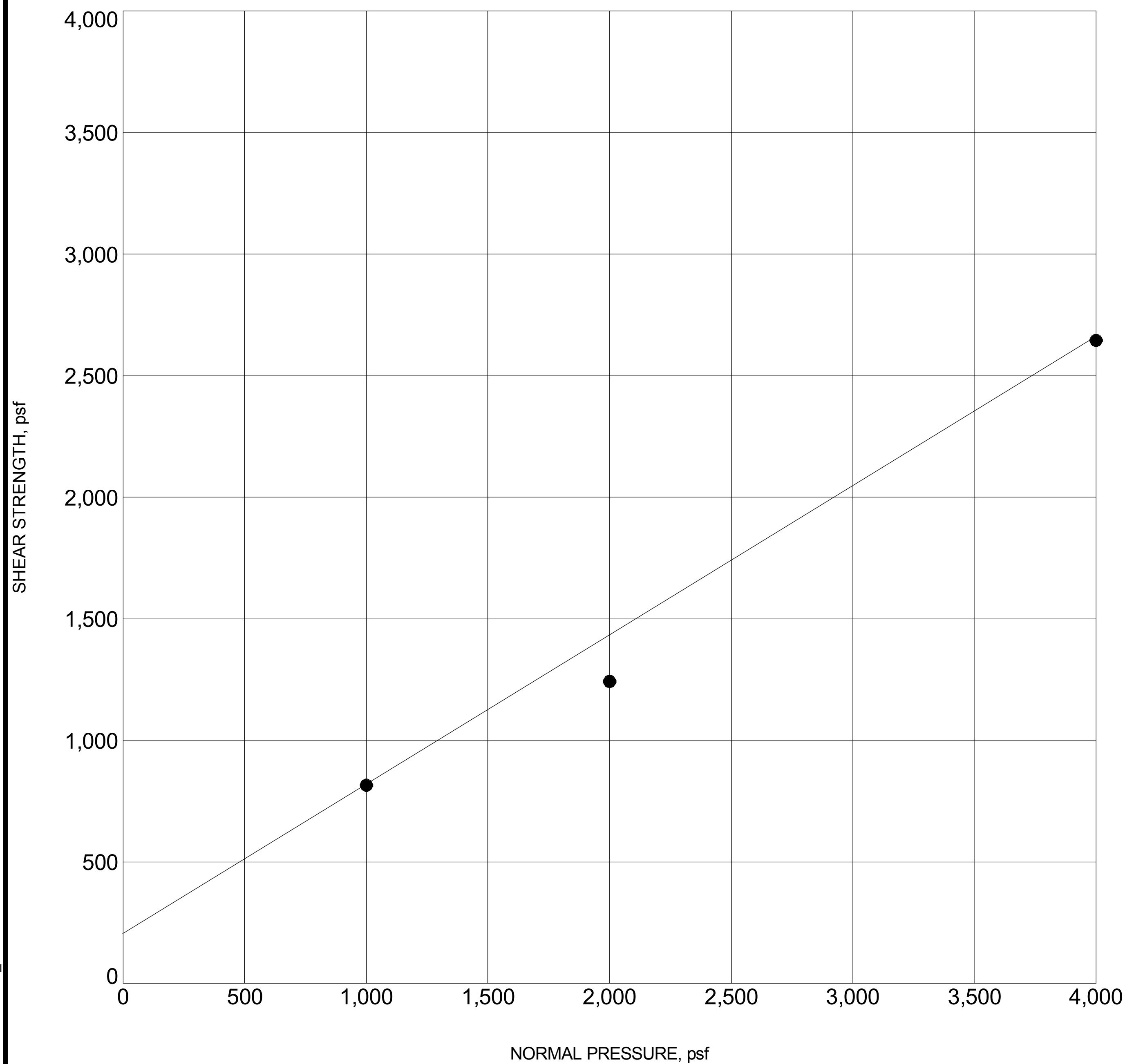
CLIENT: Morrison Maierle Inc.
Billings MT

EXHIBIT: B-4

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. DIRECT_SHEAR C4135319 MIKOSHIKA.GPJ TERRACON2012.GDT 1/31/14

DIRECT SHEAR TEST

ASTM D3080



	Specimen Identification		Classification	γ_d, pcf	WC,%	c, psf	ϕ°
●	**DH-5**	**19.5ft**	**CLAYEY SAND(SC)**	**97**	**22**	**200**	**31**

PROJECT: Makoshika State Park Road

SITE: Makoshika State Park
Glendive, MT

PROJECT NUMBER: C4135319

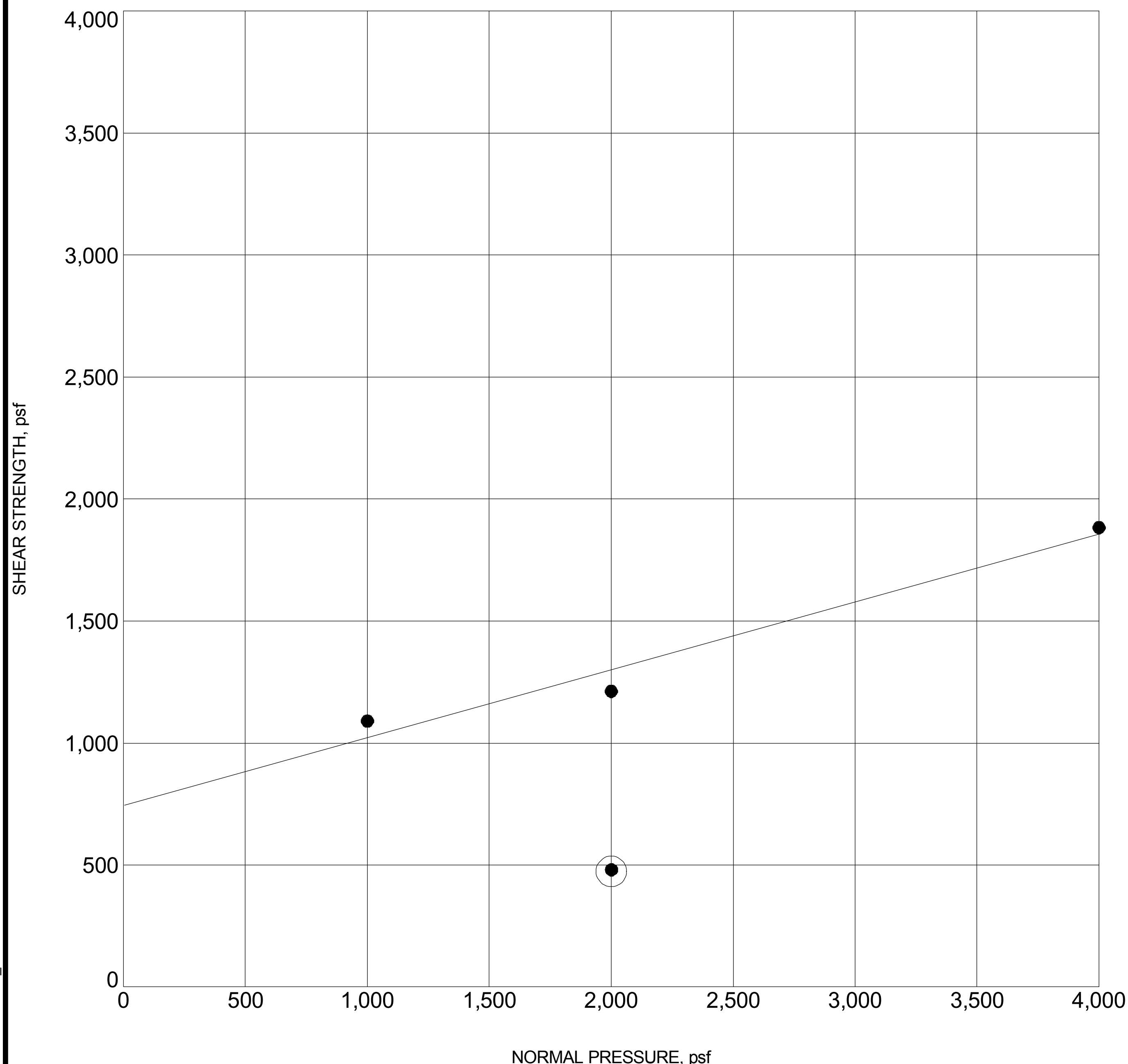
CLIENT: Morrison Maierle Inc.
Billings MT

EXHIBIT: B-5

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. DIRECT_SHEAR C4135319 MIKOSHIKA.GPJ TERRACON2012.GDT 1/31/14

DIRECT SHEAR TEST

ASTM D3080



RESIDUAL POINT

	Specimen Identification		Classification	γ_d, pcf	WC,%	c, psf	ϕ°
●	**DH-5**	**29.5ft**	**FAT CLAY(CH)**	**91**	**28**	**750**	**15**

PROJECT: Makoshika State Park Road

SITE: Makoshika State Park
Glendive, MT

Terracon

PROJECT NUMBER: C4135319

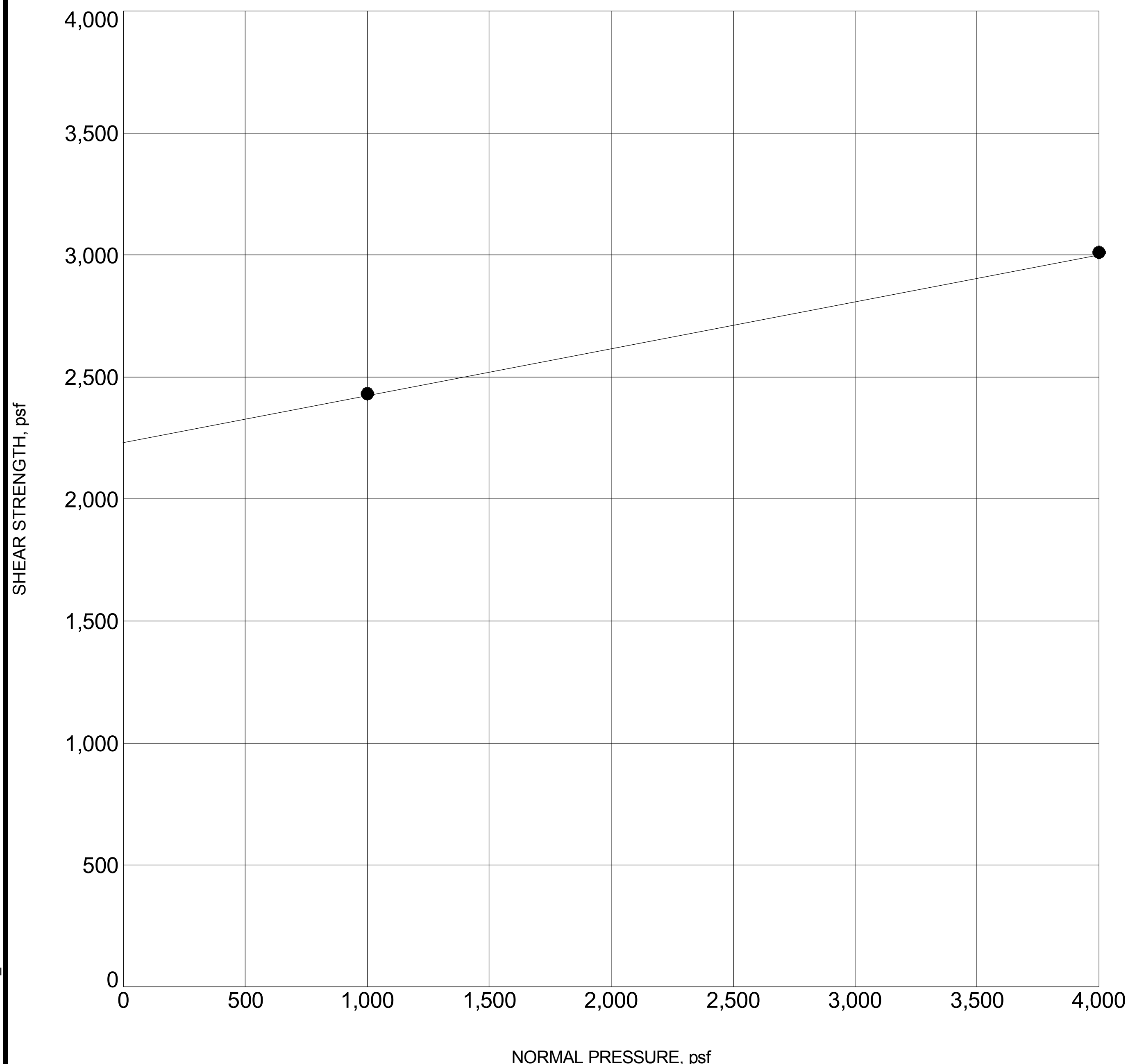
CLIENT: Morrison Maierle Inc.
Billings MT

EXHIBIT: B-6

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. DIRECT_SHEAR C4135319 MIKOSHIKA.GPJ TERRACON2012.GDT 1/31/14

DIRECT SHEAR TEST

ASTM D3080



	Specimen Identification		Classification	γ_d, pcf	WC,%	c, psf	ϕ°
●	**DH-6**	**45.0ft**	**CLAY SHALE CH**	**102**	**21**	**2250**	**11**

PROJECT: Makoshika State Park Road

SITE: Makoshika State Park
Glendive, MT

Terracon

PROJECT NUMBER: C4135319

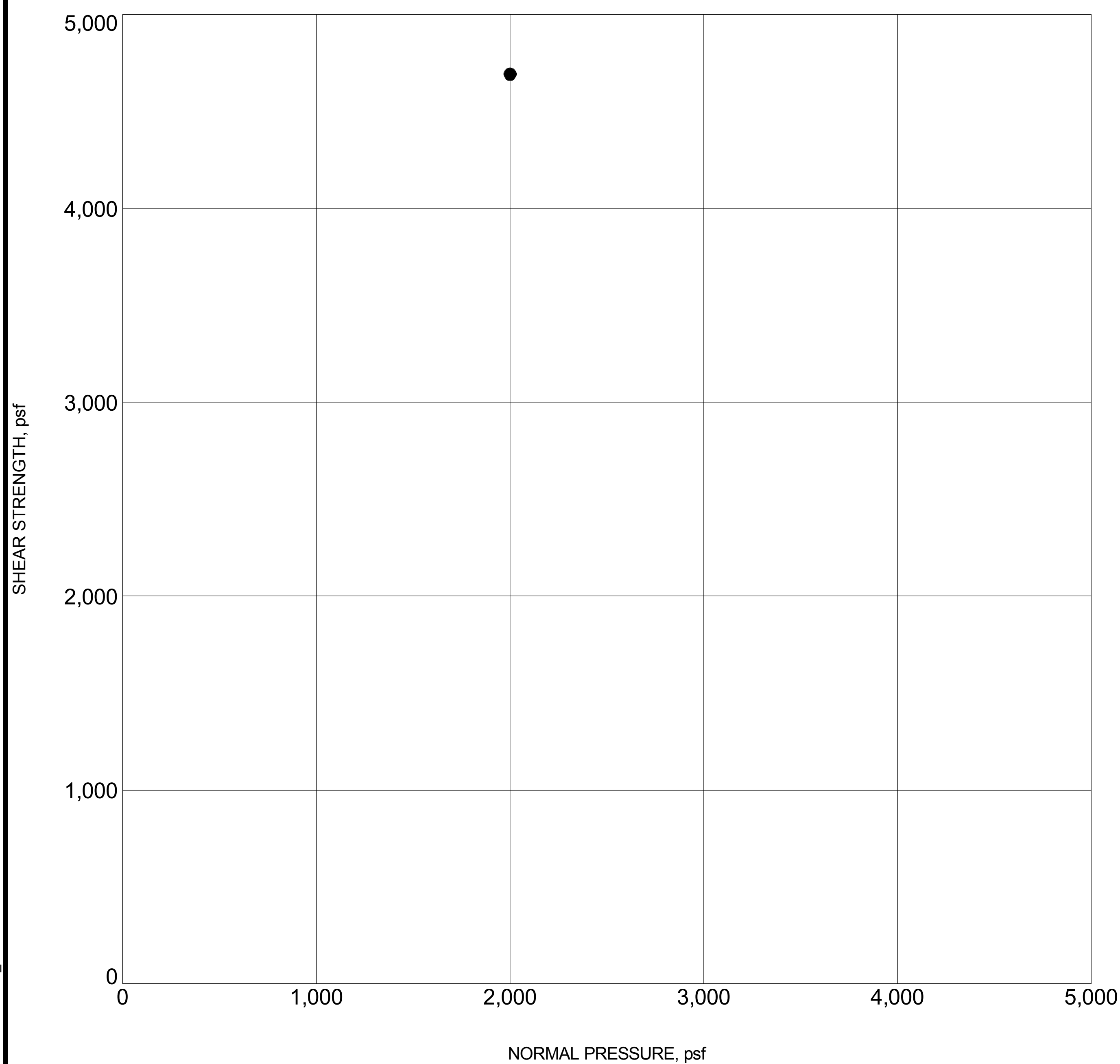
CLIENT: Morrison Maierle Inc.
Billings MT

EXHIBIT: B-7

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. DIRECT_SHEAR C4135319 MIKOSHIKA.GPJ TERRACON2012.GDT 1/31/14

DIRECT SHEAR TEST

ASTM D3080



	Specimen Identification		Classification	γ_d, pcf	WC,%	c, psf	ϕ°
●	**DH-6**	**45.1ft**	**CLAY SHALE CH**	**101**	**19**		

PROJECT: Makoshika State Park Road

SITE: Makoshika State Park
Glendive, MT

Terracon

,

PROJECT NUMBER: C4135319

CLIENT: Morrison Maierle Inc.
Billings MT

EXHIBIT: B-8

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. DIRECT_SHEAR C4135319 MIKOSHIKA.GPJ TERRACON2012.GDT 1/31/14

APPENDIX C

SUPPORTING DOCUMENTS

GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

SAMPLING

Auger	**Split Spoon**
Shelby Tube	**Macro Core**
Ring Sampler	**Rock Core**
Grab Sample	**No Recovery**

WATER LEVEL

- **Water Initially Encountered**
- **Water Level After a Specified Period of Time**
- **Water Level After a Specified Period of Time**

Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.

FIELD TESTS

(HP)	**Hand Penetrometer**
(T)	**Torvane**
(b/f)	**Standard Penetration Test (blows per foot)**
(PID)	**Photo-Ionization Detector**
(OVA)	**Organic Vapor Analyzer**

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on the Unified Soil Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

LOCATION AND ELEVATION NOTES

Unless otherwise noted, Latitude and Longitude are approximately determined using a hand-held GPS device. The accuracy of such devices is variable. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

STRENGTH TERMS

RELATIVE DENSITY OF COARSE-GRAINED SOILS (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance Includes gravels, sands and silts.			**CONSISTENCY OF FINE-GRAINED SOILS** (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance			
Descriptive Term (Density)	**Standard Penetration or N-Value Blows/Ft.**	**Ring Sampler Blows/Ft.**	**Descriptive Term (Consistency)**	**Unconfined Compressive Strength, Qu, psf**	**Standard Penetration or N-Value Blows/Ft.**	**Ring Sampler Blows/Ft.**
Very Loose	0 - 3	0 - 6	Very Soft	less than 500	0 - 1	< 3
Loose	4 - 9	7 - 18	Soft	500 to 1,000	2 - 4	3 - 4
Medium Dense	10 - 29	19 - 58	Medium-Stiff	1,000 to 2,000	4 - 8	5 - 9
Dense	30 - 50	59 - 98	Stiff	2,000 to 4,000	8 - 15	10 - 18
Very Dense	> 50	≥ 99	Very Stiff	4,000 to 8,000	15 - 30	19 - 42
			Hard	> 8,000	> 30	> 42

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 15
With	15 - 29
Modifier	> 30

GRAIN SIZE TERMINOLOGY

Major Component of Sample	Particle Size
Boulders	Over 12 in. (300 mm)
Cobbles	12 in. to 3 in. (300mm to 75mm)
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)
Sand	#4 to #200 sieve (4.75mm to 0.075mm
Silt or Clay	Passing #200 sieve (0.075mm)

RELATIVE PROPORTIONS OF FINES

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 5
With	5 - 12
Modifier	> 12

PLASTICITY DESCRIPTION

Term	Plasticity Index
Non-plastic	0
Low	1 - 10
Medium	11 - 30
High	> 30

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UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification	
				Group Symbol	**Group Name** [B]
Coarse Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$ [E]	GW	Well-graded gravel [F]
			$Cu < 4$ and/or $1 > Cc > 3$ [E]	GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH	GM	Silty gravel [F,G,H]
			Fines classify as CL or CH	GC	Clayey gravel [F,G,H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	$Cu \geq 6$ and $1 \leq Cc \leq 3$ [E]	SW	Well-graded sand [I]
			$Cu < 6$ and/or $1 > Cc > 3$ [E]	SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH	SM	Silty sand [G,H,I]
			Fines classify as CL or CH	SC	Clayey sand [G,H,I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots on or above "A" line [J]	CL	Lean clay [K,L,M]
			PI < 4 or plots below "A" line [J]	ML	Silt [K,L,M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OL	Organic clay [K,L,M,N]
					Organic silt [K,L,M,O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line	CH	Fat clay [K,L,M]
			PI plots below "A" line	MH	Elastic Silt [K,L,M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OH	Organic clay [K,L,M,P]
					Organic silt [K,L,M,Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains ≥ 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.

[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] PI ≥ 4 and plots on or above "A" line.

[O] PI < 4 or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.

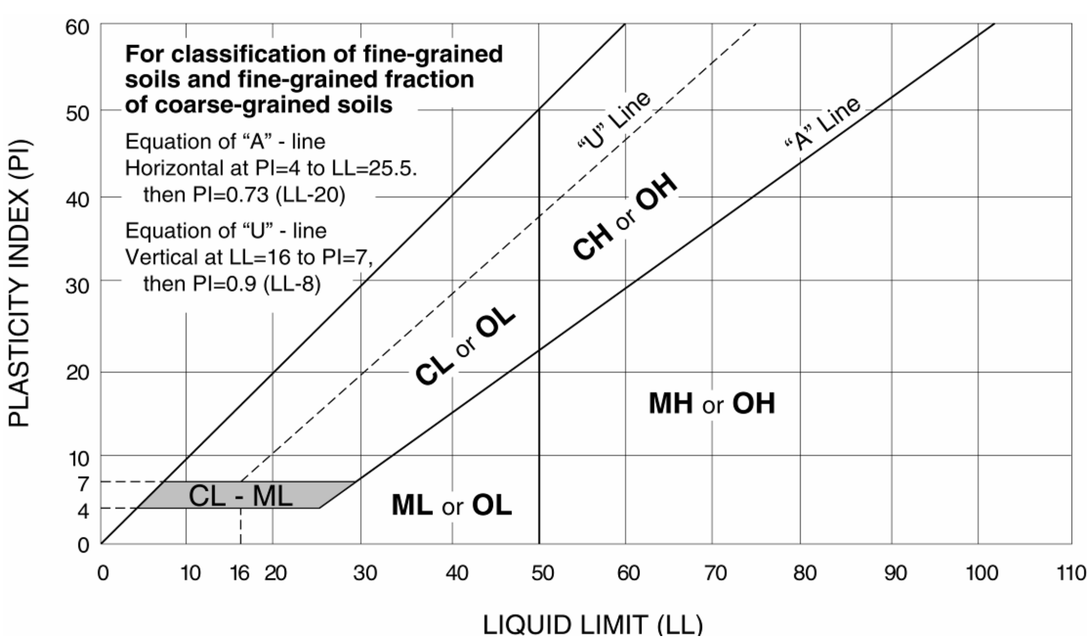

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Exhibit C-2

DESCRIPTION OF ROCK PROPERTIES

WEATHERING

Fresh	Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.
Very slight	Rock generally fresh, joints stained, some joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.
Slight	Rock generally fresh, joints stained, and discoloration extends into rock up to 1 in. Joints may contain clay. In granitoid rocks some occasional feldspar crystals are dull and discolored. Crystalline rocks ring under hammer.
Moderate	Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some show clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.
Moderately severe	All rock except quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick.
Severe	All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.
Very severe	All rock except quartz discolored or stained. Rock "fabric" discernible, but mass effectively reduced to "soil" with only fragments of strong rock remaining.
Complete	Rock reduced to "soil". Rock "fabric" not discernible or discernible only in small, scattered locations. Quartz may be present as dikes or stringers.

HARDNESS (for engineering description of rock – not to be confused with Moh's scale for minerals)

Very hard	Cannot be scratched with knife or sharp pick. Breaking of hand specimens requires several hard blows of geologist's pick.
Hard	Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.
Moderately hard	Can be scratched with knife or pick. Gouges or grooves to ¼ in. deep can be excavated by hard blow of point of a geologist's pick. Hand specimens can be detached by moderate blow.
Medium	Can be grooved or gouged 1/16 in. deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces about 1-in. maximum size by hard blows of the point of a geologist's pick.
Soft	Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure.
Very soft	Can be carved with knife. Can be excavated readily with point of pick. Pieces 1-in. or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

Joint, Bedding, and Foliation Spacing in Rock [a]

Spacing	Joints	Bedding/Foliation
Less than 2 in.	Very close	Very thin
2 in. – 1 ft.	Close	Thin
1 ft. – 3 ft.	Moderately close	Medium
3 ft. – 10 ft.	Wide	Thick
More than 10 ft.	Very wide	Very thick

a. Spacing refers to the distance normal to the planes, of the described feature, which are parallel to each other or nearly so.

Rock Quality Designator (RQD) a

RQD, as a percentage	Diagnostic description
Exceeding 90	Excellent
90 – 75	Good
75 – 50	Fair
50 – 25	Poor
Less than 25	Very poor

a. RQD (given as a percentage) = length of core in pieces 4 in. and longer/length of run.

Joint Openness Descriptors

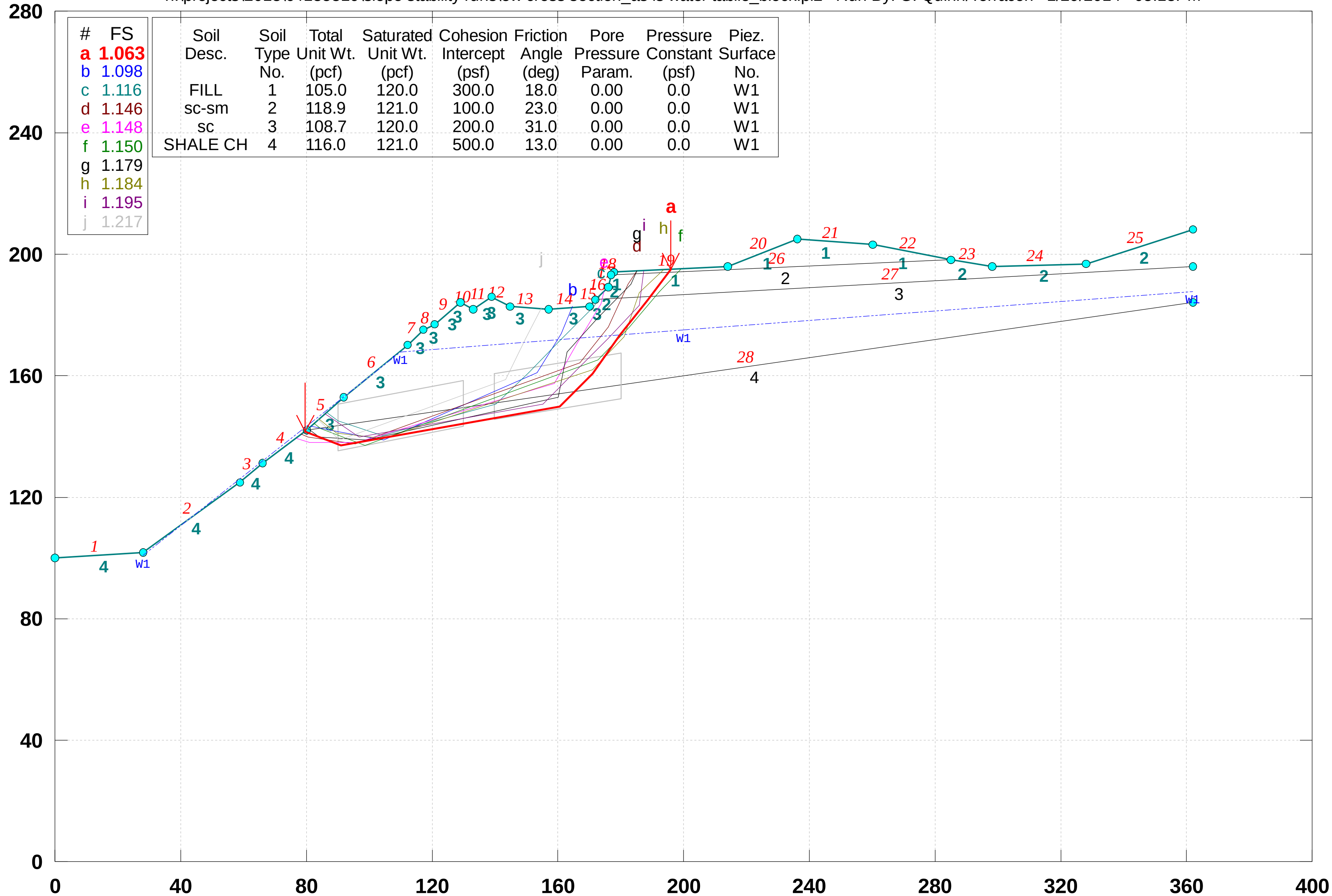
Openness	Descriptor
No Visible Separation	Tight
Less than 1/32 in.	Slightly Open
1/32 to 1/8 in.	Moderately Open
1/8 to 3/8 in.	Open
3/8 in. to 0.1 ft.	Moderately Wide
Greater than 0.1 ft.	Wide

References: American Society of Civil Engineers. Manuals and Reports on Engineering Practice - No. 56. Subsurface Investigation for Design and Construction of Foundations of Buildings. New York: American Society of Civil Engineers, 1976. U.S. Department of the Interior, Bureau of Reclamation, Engineering Geology Field Manual.

APPENDIX D
STABILITY ANALYSIS RESULTS

Makoshika 2011 Slide Existing Condition/Block Failure EXHIBIT D-1

n:\projects\2013\c4135319\slope stability runs\ew cross section_as is water tablle_block.pl2 Run By: G. Quinn/Terracon 1/29/2014 05:13PM



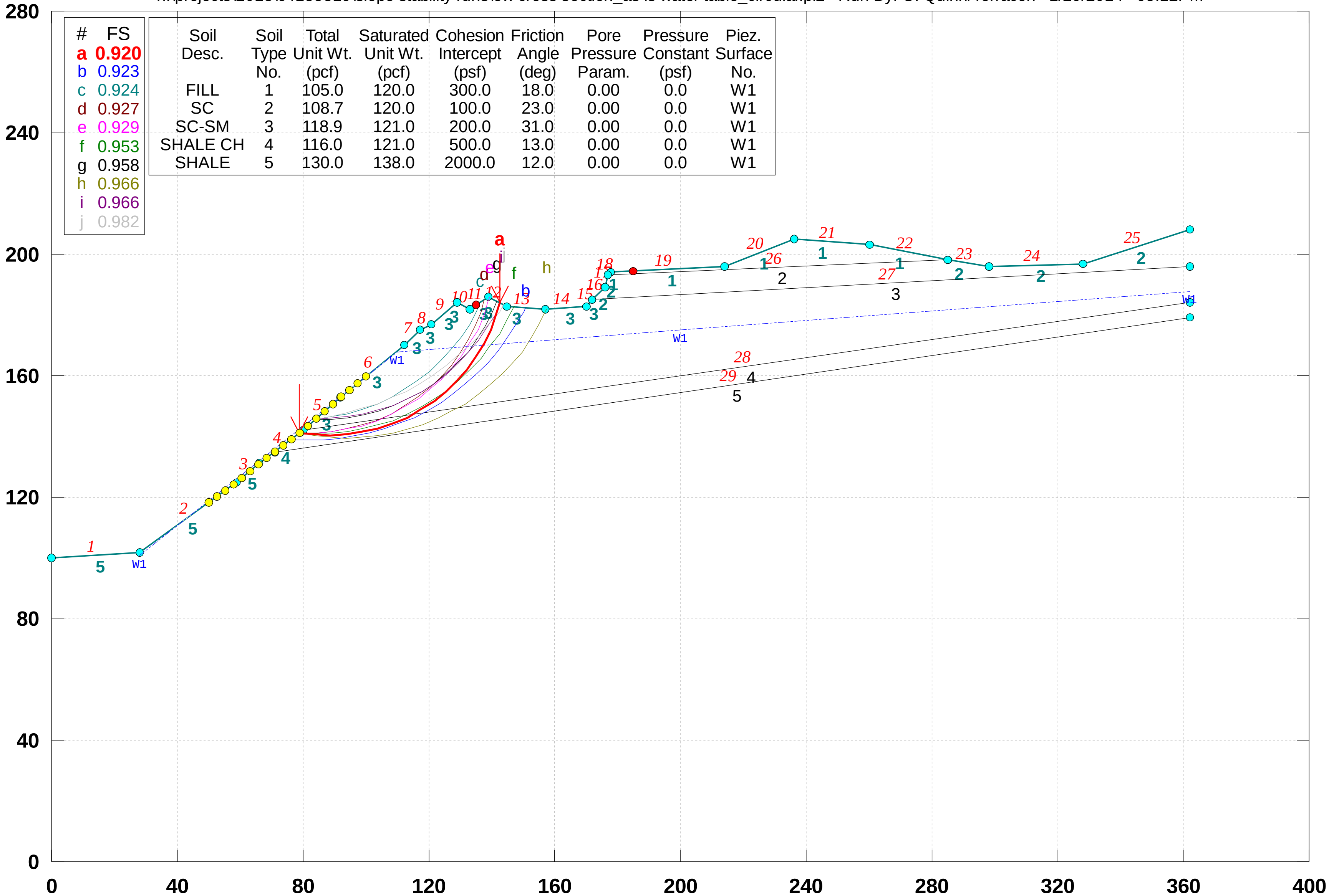
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
FILL	1	105.0	120.0	300.0	18.0	0.00	0.0	W1
sc-sm	2	118.9	121.0	100.0	23.0	0.00	0.0	W1
sc	3	108.7	120.0	200.0	31.0	0.00	0.0	W1
SHALE CH	4	116.0	121.0	500.0	13.0	0.00	0.0	W1

GSTABL7 v.2 FSmin=1.063

Safety Factors Are Calculated By The Simplified Janbu Method

Makoshika 2011 Slide Existing/ Circular Failure EXHIBIT D-2

n:\projects\2013\c4135319\slope stability runs\ew cross section_as is water table_circular.pl2 Run By: G. Quinn/Terracon 1/29/2014 05:12PM



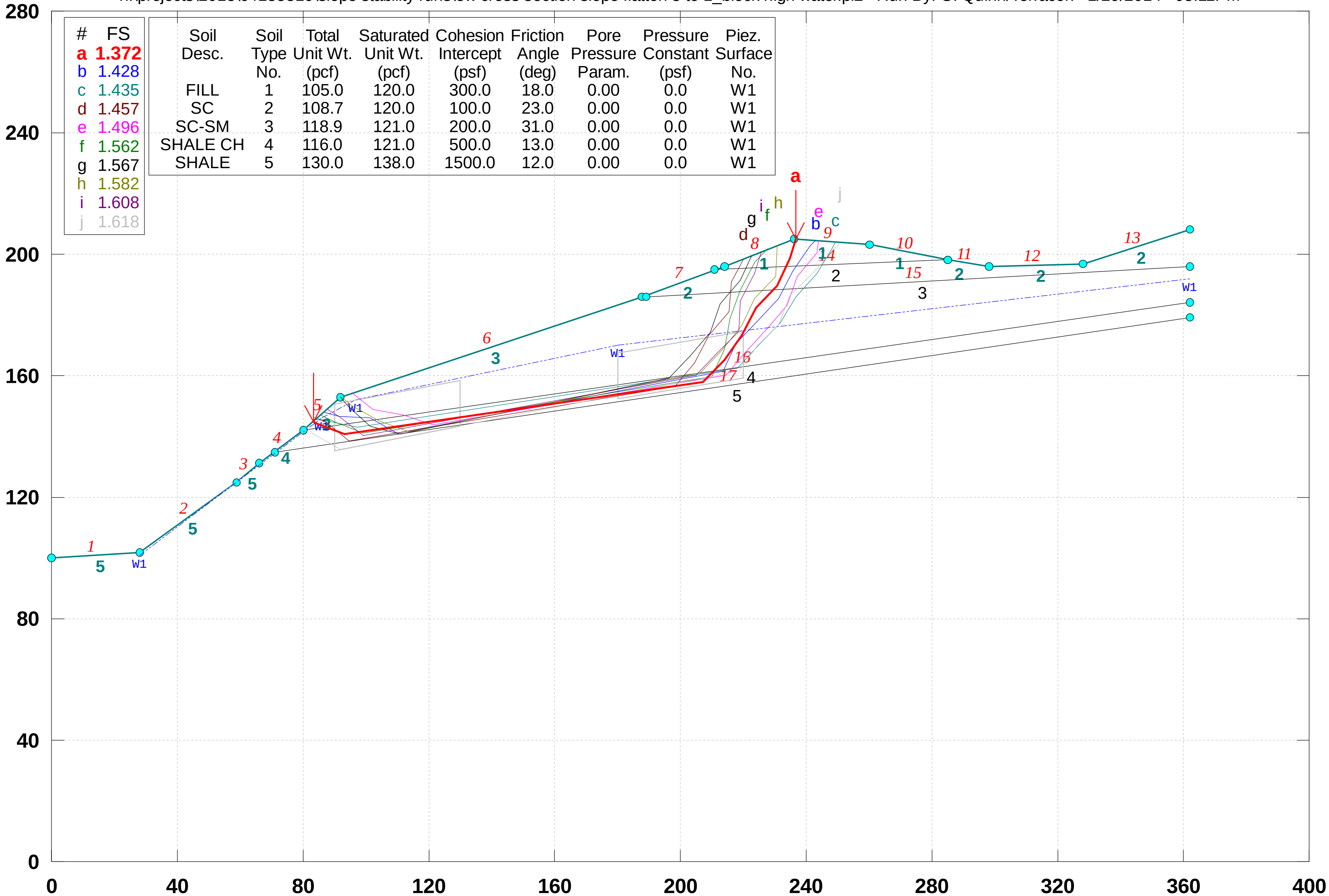
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
FILL	1	105.0	120.0	300.0	18.0	0.00	0.0	W1
SC	2	108.7	120.0	100.0	23.0	0.00	0.0	W1
SC-SM	3	118.9	121.0	200.0	31.0	0.00	0.0	W1
SHALE CH	4	116.0	121.0	500.0	13.0	0.00	0.0	W1
SHALE	5	130.0	138.0	2000.0	12.0	0.00	0.0	W1

GSTABL7 v.2 FSmin=0.920

Safety Factors Are Calculated By The Modified Bishop Method

Makoshika 2011 Slide Repair/Block 3:1 w/ Drains EXHIBIT D-3

n:\projects\2013\c4135319\slope stability runs\ew cross section slope flatten 3 to 1_block high water.pl2 Run By: G. Quinn/Terracon 1/29/2014 05:11PM

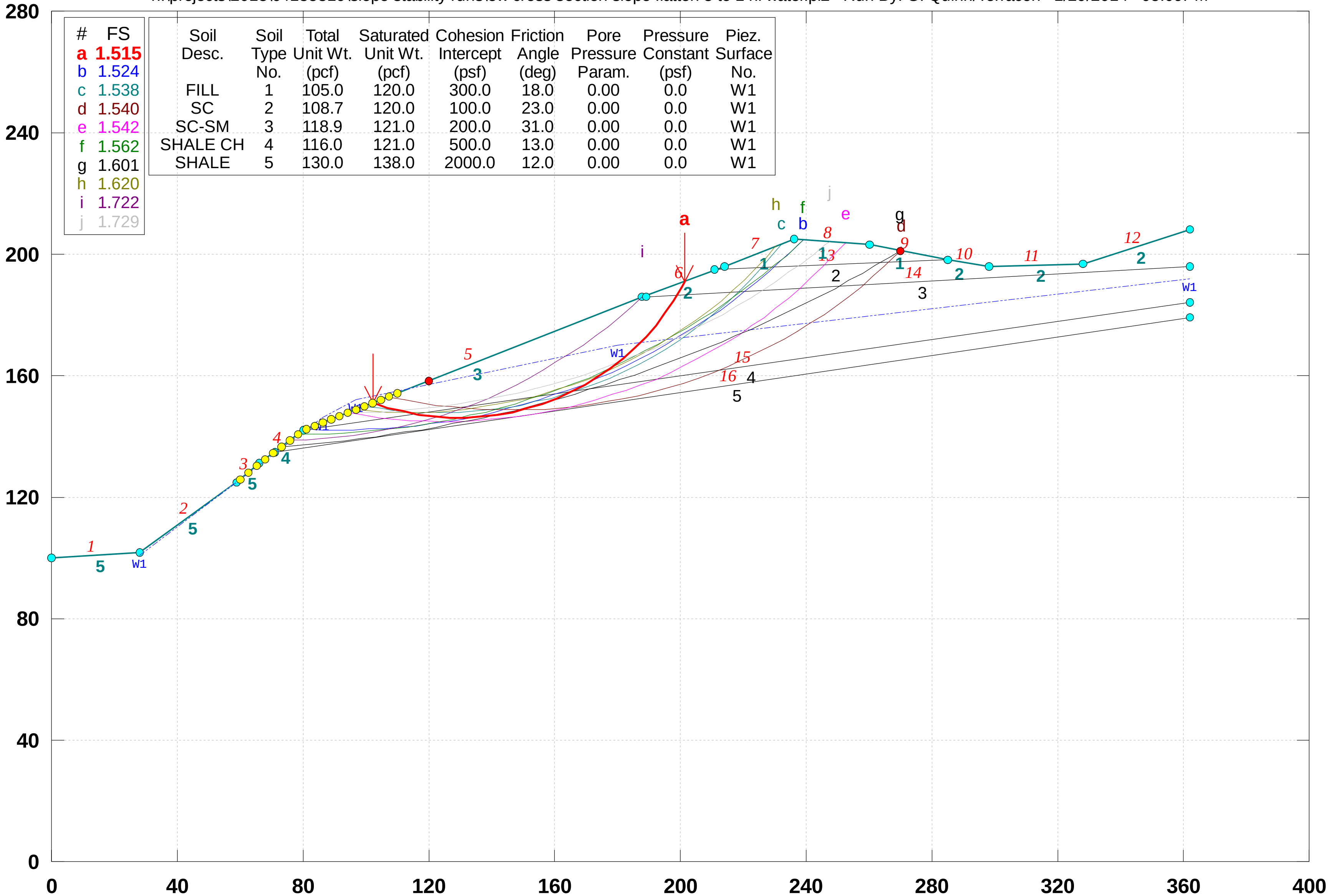


GSTABL7 v.2 FSmin=1.372

Safety Factors Are Calculated By The Simplified Janbu Method

Makoshika 2011 Slide Repair/Circular 3:1 w/ Drains EXHIBIT D-4

n:\projects\2013\c4135319\slope stability runs\ew cross section slope flatten 3 to 1 hi water.pl2 Run By: G. Quinn/Terracon 1/29/2014 05:09PM

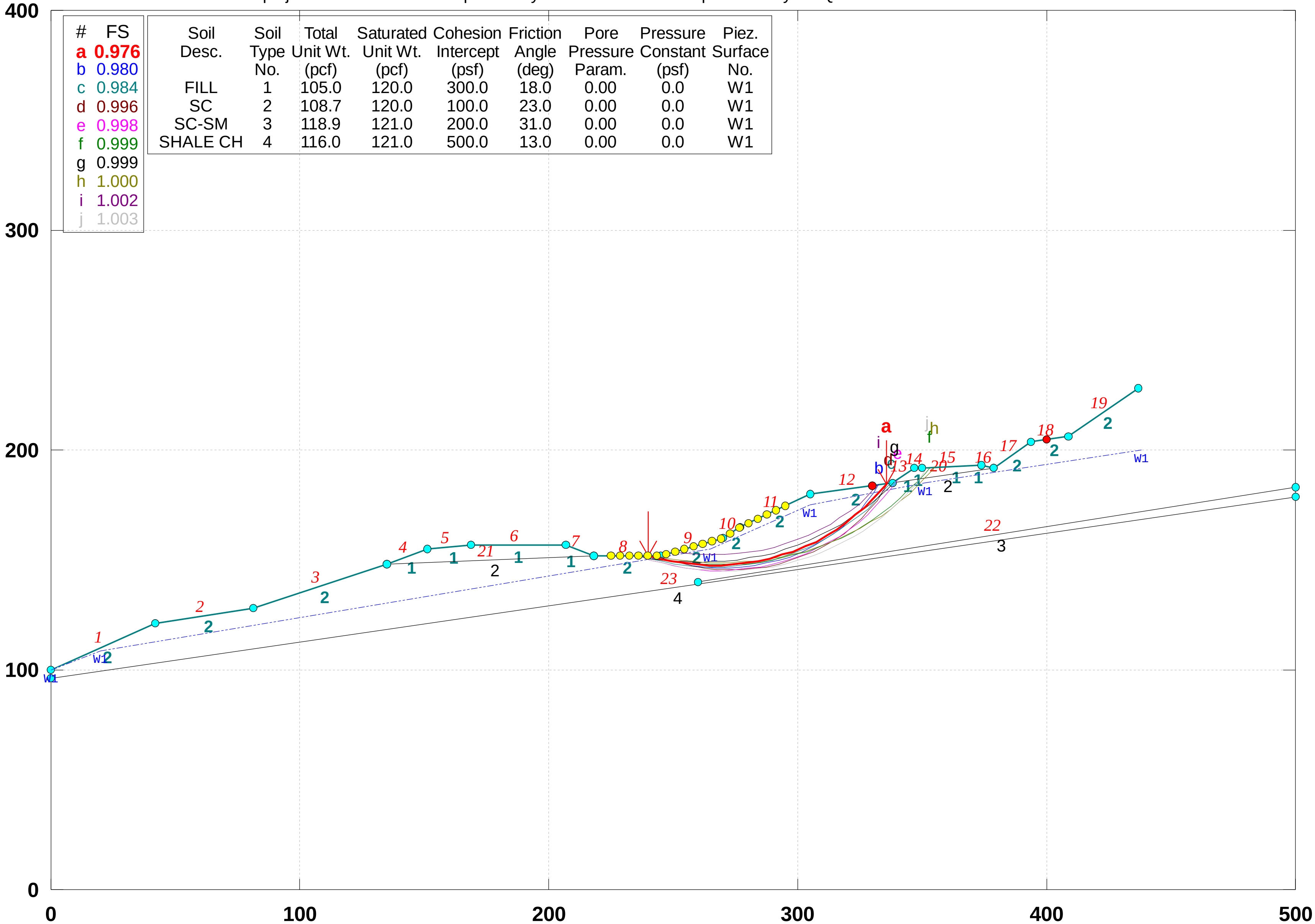


GSTABL7 v.2 FSmin=1.515

Safety Factors Are Calculated By The Modified Bishop Method

Makoshika Sta 12+00-15+00 Slide Existing Condition/Circular EXHIBIT D-5

n:\projects\2013\c4135319\slope stability runs\ns cross section.pl2   Run By: G. Quinn/Terracon   1/29/2014   05:07PM



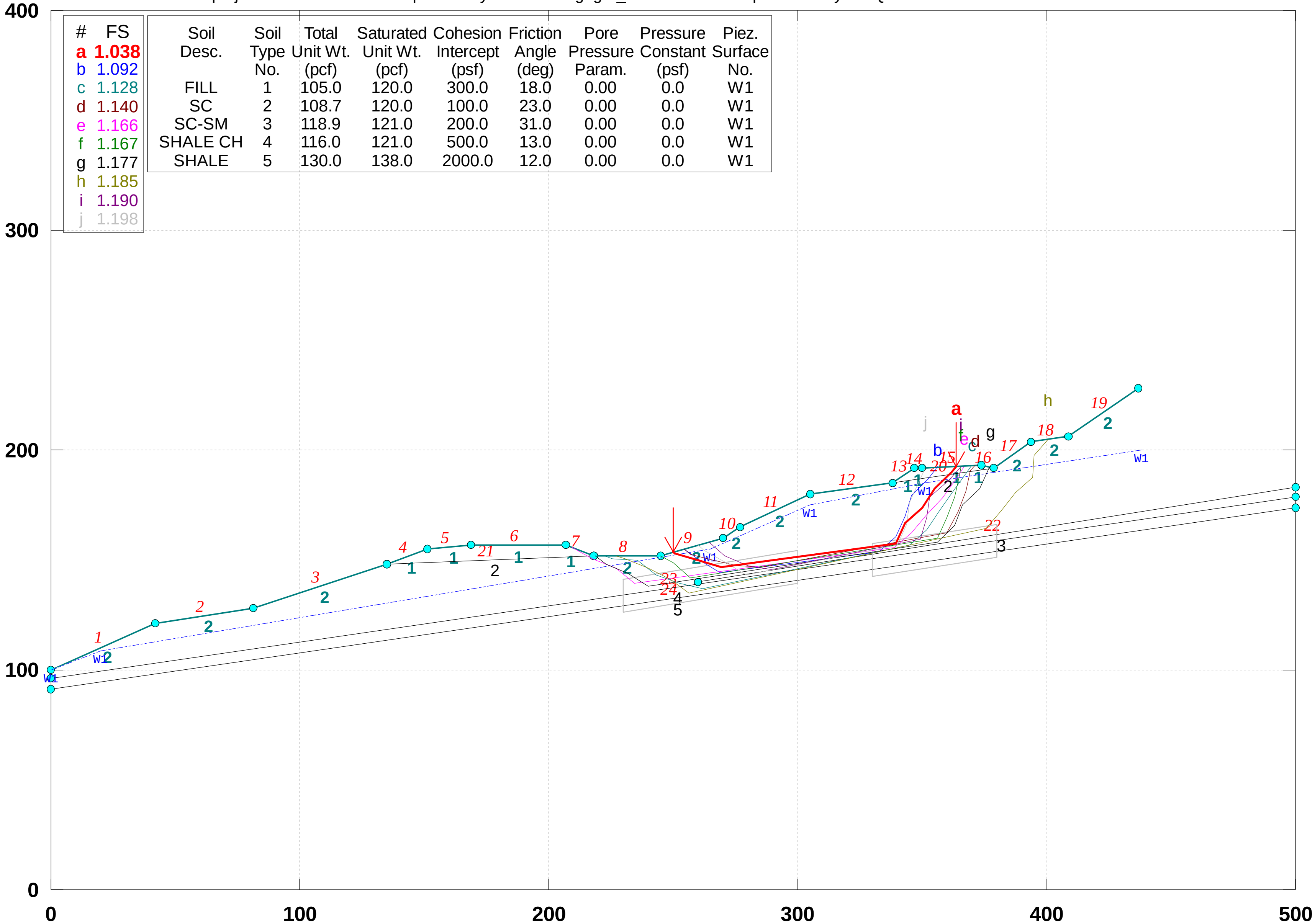
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
FILL	1	105.0	120.0	300.0	18.0	0.00	0.0	W1
SC	2	108.7	120.0	100.0	23.0	0.00	0.0	W1
SC-SM	3	118.9	121.0	200.0	31.0	0.00	0.0	W1
SHALE CH	4	116.0	121.0	500.0	13.0	0.00	0.0	W1

GSTABL7 v.2 FSmin=0.976

Safety Factors Are Calculated By The Modified Bishop Method

Makoshika Sta 12+00-15+00 Slide Existing Condition/Block EXHIBIT D-6

n:\projects\2013\c4135319\slope stability runs\existinghgwl_block without ties.pl2 Run By: G. Quinn/Terracon 1/29/2014 04:54PM



#	FS
a	1.038
b	1.092
c	1.128
d	1.140
e	1.166
f	1.167
g	1.177
h	1.185
i	1.190
j	1.198

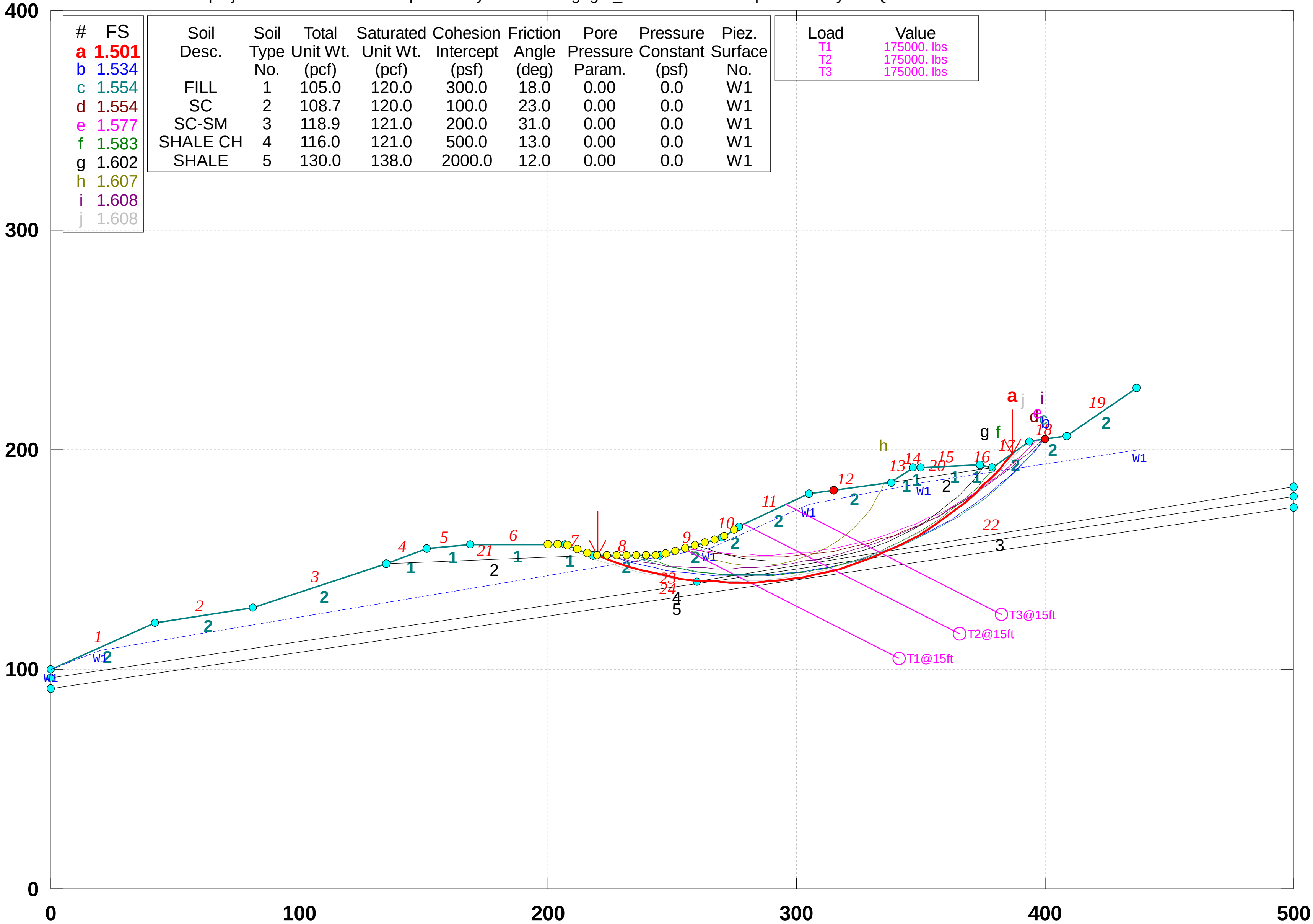
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
FILL	1	105.0	120.0	300.0	18.0	0.00	0.0	W1
SC	2	108.7	120.0	100.0	23.0	0.00	0.0	W1
SC-SM	3	118.9	121.0	200.0	31.0	0.00	0.0	W1
SHALE CH	4	116.0	121.0	500.0	13.0	0.00	0.0	W1
SHALE	5	130.0	138.0	2000.0	12.0	0.00	0.0	W1

GSTABL7 v.2 FSmin=1.038

Safety Factors Are Calculated By The Simplified Janbu Method

Makoshika Sta 12+00-15+00 Slide Repair/ Circular EXHIBIT D-7

n:\projects\2013\c4135319\slope stability runs\existinghgwl_circular with 3 ties.pl2 Run By: G. Quinn/Terracon 1/29/2014 04:44PM

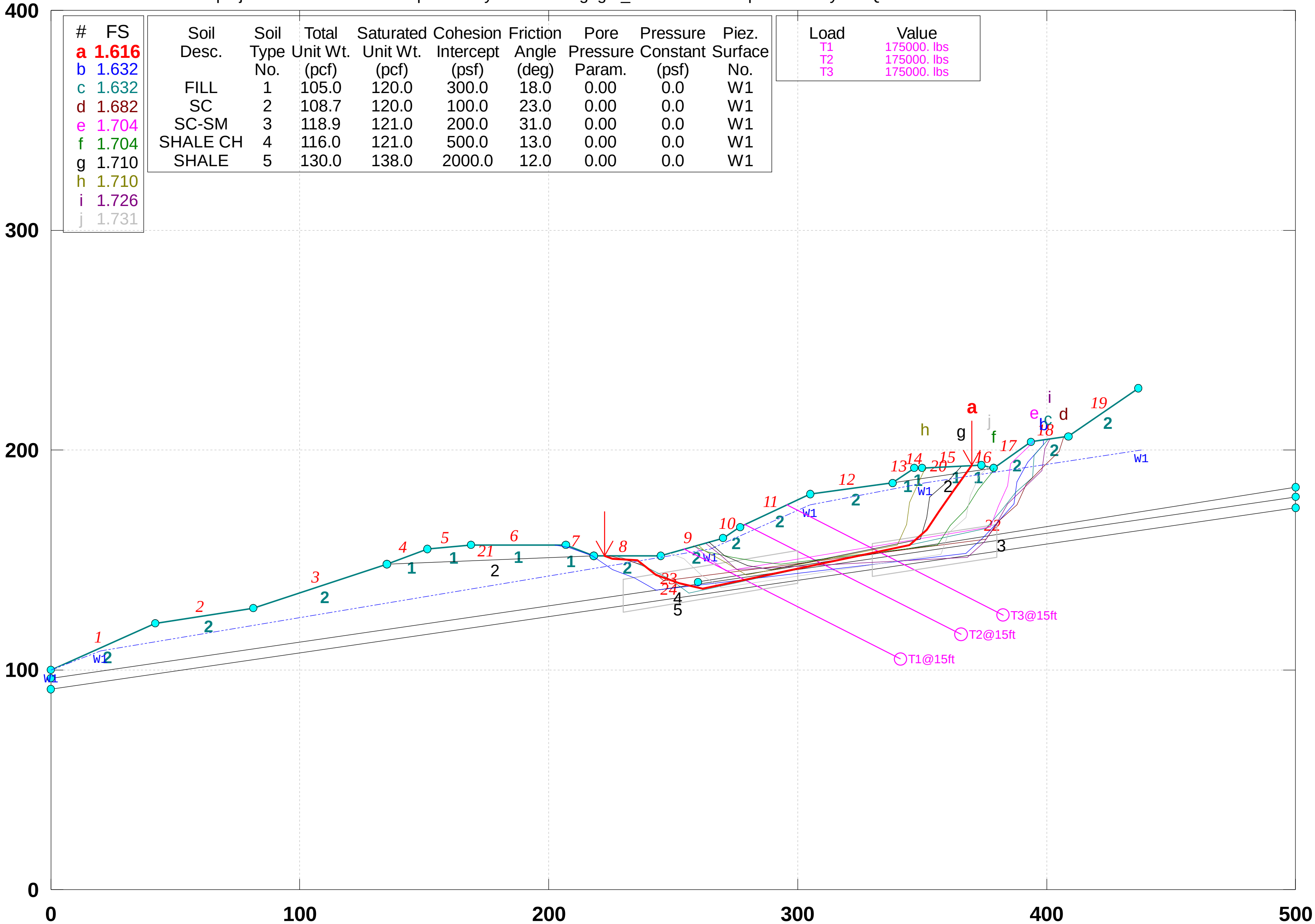


GSTABL7 v.2 FSmin=1.501

Safety Factors Are Calculated By The Modified Bishop Method

Makoshika Sta 12+00-15+00 Slide Repair/ Block EXHIBIT D-8

n:\projects\2013\c4135319\slope stability runs\existinghgwl_block with 3 ties.pl2 Run By: G. Quinn/Terracon 1/29/2014 04:24PM



#	FS
a	1.616
b	1.632
c	1.632
d	1.682
e	1.704
f	1.704
g	1.710
h	1.710
i	1.726
j	1.731

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
FILL	1	105.0	120.0	300.0	18.0	0.00	0.0	W1
SC	2	108.7	120.0	100.0	23.0	0.00	0.0	W1
SC-SM	3	118.9	121.0	200.0	31.0	0.00	0.0	W1
SHALE CH	4	116.0	121.0	500.0	13.0	0.00	0.0	W1
SHALE	5	130.0	138.0	2000.0	12.0	0.00	0.0	W1

Load	Value
T1	175000. lbs
T2	175000. lbs
T3	175000. lbs

GSTABL7 v.2 FSmin=1.616

Safety Factors Are Calculated By The Simplified Janbu Method